PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I

MAY 1953

ORDINARY MEETING

16 December, 1952

HENRY FRANCIS CRONIN, C.B.E., M.C., B.Sc.(Eng.), President, in the Chair

The President asked members to stand while he read a resolution of condolence which had been passed by the Council at their meeting that afternoon.

> "That the Council record the deep regret with which they have learned of the death of their colleague Mr Alfred Charles Gardner, F.R.S.E., who had been a member of the Institution for 43 years, and who was elected the Member of Council representing the Glasgow and West of Scotland Association for the Session 1952–1953.
>
> The Council desire that an expression of their sincere sympathy be conveyed to his family in their bereavement."

The President announced that the Council had awarded a James Alfred Ewing Gold Medal to Professor J. F. Baker, of Cambridge University. The Ewing Medal, he said, had been founded in 1936 in memory of Sir Alfred Ewing, an Honorary Member of the Institution, and the income from the endowment fund was to be expended in the provision of a gold medal to a person, whether a member of the Institution or not, for specially meritorious contributions to the science of engineering in the field of research.

Recommendations for the award of the medal were made by the Institution, the Institution of Mechanical Engineers, the Institution of Electrical Engineers, and the Institution of Naval Architects, and the medal was

awarded by the Council on the joint nomination of the President of the Institution and the President of the Royal Society.

He had very much pleasure in presenting the James Alfred Ewing Medal for 1952 to Professor Baker, who was Professor of Mechanical Science and Head of the Department of Engineering of Cambridge University since 1943, the award being made for his researches in structures.

The President then presented the Medal to Professor Baker.

The Council reported that they had recently transferred to the class of

Members

Alexander Gray, O.B.E.
Mervyn Shelley Gunasekera, B.Sc. (Eng.) (*Lond.*).
Jeffrey William Hitchen King, M.Sc. (*Manchester*).
James Robert William Murland, B.Sc. (Eng.) (*Lond.*).
Guy Richards, B.A. (*Cantab.*).
Ronald Mackay Wood, B.Sc. (*Manchester.*).

and had admitted as

Graduates

Ian Brian Adams, B.Sc. (*Nottingham*). Stud.I.C.E.
James Flett Alexander, B.Sc. (*Glas.*), Stud.I.C.E.
Norman Robert Allen, Stud.I.C.E.
Norman Edward Allender, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Allen Annesley, B.Sc. (*Belfast*), Stud. I.C.E.
Maurice Frederick George Archer, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Herbert William Argent, Stud.I.C.E.
Kanapathypilly Arumugam B.Sc. (Eng.) (*Lond.*).
Edward Cecil Ashby, B.Sc. (*Durham*), Stud.I.C.E.
John Martin Ashe, B.E. (*National*).
Albert Peter Backler, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
William Bacon, Stud.I.C.E.
Stanley Arthur Barden, B.Sc. (Eng.) (*Lond.*).
Krishan Gopal Bassi, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Alan Howard Cousens Beard, Stud. I.C.E.
Alfred Douglas Macdonald Bellis, B.Sc. (*Durham*), Stud.I.C.E.
David Alwyn Bevan, B.Sc. (Eng.) (*Lond.*).
Erik Hugh Bird, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Thomas Frederick Knowles Boucher, B.Sc. (*Belfast*), Stud.I.C.E.
Herbert Brill, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
William Kenneth Bristow, Stud.I.C.E.
Henry Bryan Parry Chapman, B.A. (*Cantab.*), Stud.I.C.E.
Thomas Edward Clarkson, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Hugh Benjamin Cochrane, Stud.I.C.E.
Vincent Knight Collinge, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
William Ian Conacher, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Bryan Walton Cooper, B.Sc. (*Birmingham*).
Charles Norman Corke, Stud.I.C.E.
John Lawrence Coulson, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
William George Cramp, B.Sc. (*St Andrews*), Stud.I.C.E.
Robert Ian Crease, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Robert James Curtis, Stud.I.C.E.
Richard Martin Custance, B.A. (*Cantab.*), Stud.I.C.E.
David Frederick Dabbs, B.Eng. (*Sheffield*), Stud.I.C.E.
Haydn Keith Davies, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
William Rhys Davies, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.

ARTHUR HUGH DAVIS, Stud.I.C.E.
MACIEJ JERZY JOZEF DEMBINSKI, B.Sc. (Eng.) (*Lond.*).
ARTHUR DOUGLAS DE VINE, B.Sc. (Eng.) (*Lond.*).
WILLIAM JAMES DICKENS, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
DENNIS EDMONDS, Stud.I.C.E.
WILLIAM HENRY ELLISON, B.Sc. (*Belfast*).
DAVID ARTHUR HOWELL EVERETT, Stud. I.C.E.
BERNARD STANLEY FARRALL, B.Sc.(Eng.) (*Lond.*), Stud.I.C.E.
NORMAN HENRY FISHER, B.Eng. (*Liverpool*), Stud.I.C.E.
DEREK EDWARD FITTES, B.Sc. (*Durham*), Stud.I.C.E.
MICHAEL ELLIOT FITZGIBBON, B.A., B.A.I. (*Dublin*).
PETER JOHN FOX, B.Sc. (Eng.) (*Lond.*).
DESMOND FOSTER SEPTIMUS GAMLEN, B.Sc. (*Nottingham*), Stud.I.C.E.
DAVID HOSKINS GIDDY, B.Sc. (*Witwatersrand*).
FREDERICK GILMAN, Stud.I.C.E.
GRAHAM ARTHUR GOLLAN, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
CHARLES HARVEY GOLSON, B.Sc. (*Cape Town*), Stud.I.C.E.
DAVID FARRAND GOODMAN, B.Eng. (*Liverpool*), Stud.I.C.E.
JAMES DOUGLAS GRAHAM, B.Sc. (*Glas.*), Stud.I.C.E.
HARVEY JOHN GUDGE, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
JOHN RUSSELL HENDERSON, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
BRIAN HEYS, M.A., Ph.D. (*Cantab.*).
DAVID ATHOLL HISLOP, B.Sc. (*Glas.*).
GEORGE STUBBS HODGSON, B.Sc. (*St Andrews*), Stud.I.C.E.
FRANK HOLROYD, B.Eng. (*Liverpool*), Stud.I.C.E.
JOHN HOLT, B.Sc. (*Leeds*), Stud.I.C.E.
DONALD ARTHUR JACKSON, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
NEIL JACKSON, B.Sc. (*Leeds*), Stud.I.C.E.
JOHN MAURICE JAGGER, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
GRAEME FREDERICK JAMES, B.E. (*New Zealand*).
DONALD SUTCLIFFE JENNINGS, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
ALAN STANLEY JOHNSON, B.Sc. (Eng.) (*Lond.*).
ALASTAIR WILLIAM JOHNSON, B.Sc. (*Natal*).
DONALD WALTER LESLIE ROY JOHNSON.
ERIC DE LLOYD JONES, B.Sc. (*Wales*), Stud.I.C.E.
IVOR PHILIP THOMPSON JONES, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
RUPERT BERNARD KERI JONES, B.Sc. (*Wales*), Stud.I.C.E.
MAURICE PETER KNELL, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
JOHN LANGDON, B.Sc.Tech. (*Manchester*).
JOHN PHILIP LAPPIN, Stud.I.C.E.
DESMOND KANE LAWSON, B.Sc. (*Belfast*), Stud.I.C.E.
DAVID JOHN LEE, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
ROBERT LEE, B.Sc. (*Manchester*), Stud. I.C.E.
WILLIAM ANDERSON LEE, B.Sc. (*Glas.*), Stud.I.C.E.
ARCHIBALD JOHN LEGG, Stud.I.C.E.
JOHN HAROLD LETHBRIDGE, B.Sc. (Eng. (*Lond.*), Stud.I.C.E.
DONALD JOHN LEVELL, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
ROBERT UNDERWOOD LEWIS, Stud. I.C.E.
LESLIE LIPWICK, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
JOHN LONERGAN, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
COLIN JEFFREY NEWTON LOWE, B.Eng. (*Liverpool*).
RONALD BRYAN LYNSKEY, B.Eng. (*Liverpool*).
HENRY MCCALLION, B.Sc. (Eng.) (*Lond.*). Stud.I.C.E.
ROBERT STEVEN MACGREGOR, B.Sc. (*St Andrews*).
SOLOMON DAVID MARGOLIS, B.Sc. (*Cape Town*), Stud.I.C.E.
BRIAN MARSH, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
PETER LOUIS ANTHONY MARSH, B.Sc. (*Cape Town*), Stud.I.C.E.
ARTHUR MARSLAND, B.Sc. (*Manchester*), Stud.I.C.E.
LAURENCE HAROLD MARTIN, B.Sc. (*Leeds*), Stud.I.C.E.
JOHN WILLIAM MELROSE, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
ROBERT PETER MELSON, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
GORDON ARCHIBALD IAN MILLAR, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
JOSEPH HYACINTH MITCHELL, B.E. (*National*), Stud.I.C.E.
KENNETH ROBERT MITCHELL, B.Eng. (*Liverpool*), Stud.I.C.E.
JOHN HERBERT MOORES, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
ALAN GARNESS MORRIS, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
JAMES FYFFE MORRISON, Stud.I.C.E.
JAMES CYRIL MUGGLESTONE, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
NORMAN WILLIAM MUNDAY, B.Sc. (*Belfast*), Stud.I.C.E.

Charles Henry Homan Murphy, B.A., B.A.I. (*Dublin*).
Frederick Harold Needham, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Brian Leonard Nicholls, Stud.I.C.E.
Michael John Ronald Sidney Norman, B.Sc. (Eng.) (*Lond.*), Stud I.C.E.
Patrick James Norton, Stud.I.C.E.
David Nicholson Oliphant, B.Sc. (*Edin.*), Stud.I.C.E.
Edward Daniel O'Reilly, B.Sc. (Eng.) (*Lond.*).
Allan John Owen, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Girvan Meir Parton, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Alan Husband Parvin, Stud.I.C.E.
John Patience, B.Sc. (*Edin.*).
Thomas Stephen Peate, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Cyril John Peniket, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
James Denvers Eustace Perera, B.Sc. (Eng.) (*Lond.*),
John Peverel-Cooper, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Eric Pickles, B.Sc. (*Durham*), Stud. I.C.E.
Mellor Procter Poucher, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Robert John Pryor, B.Eng. (*Sheffield*), Stud.I.C.E.
Phelim Quinn, B.Sc. (*Belfast*).
Diarmuid Francis Quirke, B.E. (*National*).
Bernard Percy Reynolds, B.Sc. (*Birmingham*), Stud.I.C.E.
John Stringer Reynolds, B.Sc. (*Nottingham*).
Bruce Harold Rowden, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
William Robert Roycroft, B.E. (*National*), Stud.I.C.E.
John William Raymond Rumsey, B.Sc. (Eng.) (Lond.), Stud.I.C.E.
Patrick Cyril Sadler, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Andre Pierre Savoie, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Emmanuel Thiruchelvam Savundranayagam, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
Paul Herbert Scarlett, B.Sc. (*Belfast*), Stud.I.C.E.
Henry Ivan Schwartz, B.Sc. (*Witwatersrand*).
Shaoul Gourgi Shahrabani, Stud.I.C.E.
Richard Hartley Shaw, B.Sc. (*Manchester*), Stud.I.C.E.
Terence James Gillespie Simms, B.Sc. (*Belfast*), Stud.I.C.E.
Ralph Berry Sims, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Geoffrey Charles Skinner, Stud.I.C.E.
Desmond Evelyn Smith, B.Sc. (*St Andrews*), Stud.I.C.E.
Geoffrey Ernest Smith, B.Sc. (Eng.) (*Lond*), Stud I.C.E.
Harold Geoffrey Smith, Stud.I.C.E.
Leslie Derek Smith, B.Sc. (Eng.) (*Lond.*) Stud.I.C.E.
James Archibald Gordon Sneddon, Stud.I.C.E.
Neville Allan Soper, B.Sc. (*Birmingham*), Stud.I.C.E.
Fredric Charles Speldewinde, Stud. I.C.E.
Julian Erwin Spindel, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Bruce McKenzie Swanson, B.E. (*New Zealand*), Stud.I.C.E.
Alan George Tate, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
John Brian Tattam, B.Sc. (Eng.) (*Lond.*) Stud.I.C.E.
Robert White Taylor, B.Sc. (*Glas.*), Stud.I.C.E.
Roy Taylor, B.Sc. (*Nottingham*), Stud. I.C.E.
Neil Lawton Thomas, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Allan Gordon Tomlin, B.Sc. (Eng.) (*Lond.*)
Albert Toothill, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Arthur George Townsend, B.Sc. (*Belfast*), Stud.I.C.E.
John Trevor Turrill, B.Sc. (*Nottingham*),
Frederick Leonard Tutt, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Ernest Charles Venes, B.Sc. (Eng.) (*Lond.*).
John Walsh, B.Sc. (*Nottingham*), Stud. I.C.E.
Antony John Waters, B.Sc. (*Birmingham*), Stud.I.C.E.
Thilakaratne Wijesingne, B.Sc. (Eng.) (*Lond.*).
David Alan Wilcock, B.Sc. (*Durham*), Stud.I.C.E.
Norman Victor Willcocks, B.Sc. (Eng.) (*Lond.*).
William Richard Winnett, B.A. (*Cantab.*).
Eric Albert Wrigley, Stud.I.C.E.
Frank Paul Zimmermann, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.

and has admitted as

Students

James Adams.
David Julius Adler.
George Graham Anthony.
Trevor Bailey.
Peter John Baker.
Michael Gordon Bennett.
John Arthur Bergg.
William Rupert Blain
Ashley Herbert Boddy.
John Philip Boydell.
Ronald Ernest Brand.
William Trevor Dennis Brown.
James Burgess.
Derek Peter Burgoyne.
David Charles Burman.
Alec Clifford Carpenter.
Terry Chadwick.
Fredrick James Chestnutt.
Walter Campbell Cleland.
Nigel David Clifford.
Paul Anthony Clifford.
Richard Derek Coleman
Michael Jim Conacher.
Brian Arthur Cook.
David Macleod Cook.
Graham David Cook.
Peter Francis Cook.
Richard Minto Costain.
James Anderson Cowper.
David Joseph Daniel.
Dinshaw Jehangir Daruvala.
John Weir Davidson.
Glynne Vaughan Davies.
Sembukuttige Henry Cornelius de Silva.
Parkash Chand Dhuna.
Victor Januarius Peter Dias.
William Warden Dick.
William Downie.
Duncan Roland Drake.
Barry Craig Eccleston.
Francis William Emerson.
Oscar Roy Ericsson.
David Royston Evans.
Harold Cecil Fennell.
Kenneth Alfred Furness.
John Fleming Gavin.
John Noel Gibson.
Roger Cedric Glasbey.
Donald Leslie Gudgeon.
Edward Blackwood Hall.
George Douglas Hamilton.
George Graham Harris.
Jack Harrop.
Brian Peter Haskins.
Peter Rushworth Hirst.
Derek Caesar Hurden.
Bertram Lawrance Hurst.
Michael Arthur Jackson.
John Franklin Jones.
Neville Herbert Jones.
Peter Edward Joselin.
Brian William Kitching.
James Anders John Knox.
John Peter Lamont.
Eric William Lang.
Francis Denis Light.
Leszek Lukomski.
John Miller McCarte.
Alfred McGill.
George Osborne Francies McLean.
William Stephen Mallagh.
Leonard Acford Marrs.
John Geoffrey Measor.
Alex Francis Mark Mendoza.
Howell Richard Merchant.
Harold Wilfred Miller.
Colin Milligan.
Herbert Wilfrid Milsom.
Kenneth Alan Minty.
Desmond Moore.
Christopher John Morriss.
Michael James Moseley.
Jonathan Mountain.
Matthew William Mullins.
Harry Morton Neal.
Thomas Alphonsus O'Driscoll.
Olusoji Olugbekan.
James Murray Reid Orr.
Geoffrey Michael Payne.
Colin James Peters.
Maurice Garfield Porter.
Ralph Pugh.
Peter Grant Pyves.
Arthur David Rigby.
Ian Dudley Robson.
William Rowe.
Austen Geoffrey Shrives.
Michael John Stedman.
Gerald Stuart Tarte.
Vivian Edward Thomas.
John Peter Thomson.
John Hughes Tilley.
Matthew Ian Tremain.
Richard Todd Trigg.
Peter Donald Westwood.
John Oates Widger.
Michael John Farquhar Willett.
Barry Ian Willson.
James Wright.
John Lloyd Wynne.
David Ivor Bailey Young.

THE UNWIN MEMORIAL LECTURE, 1952

The President reminded members that, in 1947, Miss E. T. Unwin, a niece of the late Dr W. C. Unwin, Past-President of the Institution, bequeathed a sum of £2,500 to the Institution. The conditions attaching to this legacy were that the sum of £1,500 was to be invested in trust for the foundation of a " William Cawthorne Unwin Lectureship " consisting of an Unwin Memorial Lecture on Engineering Research to be delivered to the members of the Institution annually, or at such other period of recurrence as the Council might determine. The remainder of the legacy was for the endowment of a section of the Institution Library.

The Council had this year requested Mr T. M. Herbert, Director of Research, the Railway Executive, to deliver the fourth Unwin Memorial Lecture.

Mr Herbert then delivered the following Lecture.

" The Development and Functions of the Research Department of the Railway Executive "

ANYONE who is honoured by an invitation to deliver a memorial lecture must seek to emphasize some particular facet of his subject's character or achievement, and to find some link between him and the subject of the lecture. The task of the Unwin Memorial lecturer is greatly facilitated by the existence of Mr E. G. Walker's excellent and fascinating account of Unwin's life and work, and after reading it I believe that the appropriateness of my subject, which was commended to your Council by Mr J. C. L. Train, M.C., M.I.C.E., is not in doubt. Unwin's work and interests showed a breadth and versatility that were a characteristic of a research organization serving the needs of a railway ; and it is interesting to recall that during his early association with Sir William Fairbairn, he carried out two pieces of research on railway problems : one to assess the relative merits of two forms of continuous brake for railway trains, the other on the design and safety of wrought-iron railway bridges. He himself, in his presidential address to Section G of the British Association in 1892, used words which contain a justification for the existence of any Research Department :—

> " There are, no doubt, some people who are in the habit of depreciating quantitative investigation of this kind. They are as wise as if they recommended a manufacturer to carry on his business without attending to his account books. Further, the attempt to obtain any clear guidance from experiments in steam engines has proved a hopeless failure, without guidance from the more careful scientific analysis.

There is not a fundamental practical question about the thermal action of the steam engine, neither the action of the jackets, nor of expansion, nor of multiple cylinders, as to which contradictory results have not been arrived at, by persons attempting to deduce results from the mass of engine tests without any clear scientific knowledge of the conditions which have affected particular results. In complex questions fundamental principles are essential in disentangling the results. Interpreted by what is already known of thermodynamic actions, there are very few trustworthy engine tests which do not fall into a perfectly intelligible order."

I have naturally studied, also, at the outset of my task, the three most eloquent Lectures delivered by my predecessors, Dr Oscar Faber, Professor Pippard, and Professor Evans. I noted with some alarm the almost uniform rate at which successive Unwin Lectures increased in length, but I trust I may be allowed, if only in the interest of those who follow me, to curb this somewhat alarming trend.

My title—The Development and Functions of the Research Department of the Railway Executive—is at first sight a little strange, but it permits me at the outset to mention two diametrically opposed opinions which are not infrequently expressed. On the one hand there are those who would regard the railway industry as so traditional and well-established that there can now be little scope for research. The other view is that it is in just such an industry that research, by some magic wand, can produce revolutionary changes and improvements overnight. I trust that in my Lecture I may be able to demonstrate the falseness of both these opinions.

Functions of a Railway Research Department

Need and Justification

It will not be out of place to give some consideration to the functions of railways in an attempt to define the scope for research in the railway industry. Railways exist to transport passengers and freight as safely, cheaply, and efficiently as possible. They are fundamentally a secondary industry—a commercial service—and it is only as an essential to providing this service that they are a vast technical undertaking as well. It is interesting to record that in 1951 the receipts of British Railways amounted to £373 million and the expenditure to £338 million—showing a surplus of £35 million: of the expenditure, about two-thirds went in operating and other traffic costs, and one-third in the maintenance and depreciation of the rolling stock, permanent way, and structures.

Both the technical and the even larger non-technical branches of the railway industry give rise to problems in the solution of which, to a greater or less extent, scientific research, or at any rate the scientific method of approach, can play a part. It would be wrong, however, to suggest that

the solution of all such problems could ever be effectively delegated exclusively to a research department, however extensive or well-equipped. It would be impracticable, and indeed fatal, to deprive the individual operating, commercial, or technical officer of the responsibility and opportunity for progress and development. A major function of a research department, as I see it, is to assist in this work by means of the special equipment and techniques which modern science has made available, and with which a scientifically trained staff is familiar. Only a limited number of problems, or phases of a problem, are suitable for handling exclusively in the Research Laboratory, although a much greater proportion is likely to be solved more convincingly if scientific advice is invoked both in the planning of experiments and in the making and analysing of observations.

It will be generally accepted that development and research in methods of railway operation in its broadest sense can only be effectively initiated, in the main, by those in day-to-day contact and control of the undertaking, aided whenever possible by such operational and scientific research methods as can contribute to the study of their diverse problems. It is, however, sometimes argued that research directed towards improving the technical equipment of railways, and the great variety of products which they consume, could more effectively be left to the suppliers of that equipment and those materials. It is contended in support of this view that most railways purchase their requirements from industry, and whilst on the locomotive and rolling-stock side this is not true of Great Britain to the extent that it is in other countries, even British Railways necessarily utilize a vast array of equipment and materials purchased from outside firms, much of which is not specifically of a railway character. What, then, is the justification for British Railways to equip itself with considerable research facilities of its own ?

There are, I think, three good reasons for the policy of British Railways in maintaining its own research organization. First, it is important to emphasize a fundamental difference in outlook, which in turn affects the scope for research, between the railways and, for example, private manufacturers of locomotives and rolling stock. The manufacturer's aim is to sell his products, and although he is, of course, concerned that his products should give a good account of themselves, he is not, unlike the railways, responsible for subsequent performance and maintenance. Whilst, therefore, both the railways and manufacturers are interested in research directed to improved design and lower first cost, the railways are equally concerned with research relating to performance and to the reduction in maintenance costs. This distinction tends to emphasize, on the part of the railways' research programme, the importance of research on materials, wear, corrosion, etc.

In other words, even if a railway buys all its equipment and materials from industry, it still has to use and maintain that equipment, usually under the rather special and often arduous conditions of railway service,

with a responsibility for safety and reliability which it cannot delegate. It alone knows, or can get to know, exactly what these conditions of service are ; it alone can readily study the behaviour of its equipment and materials under everyday conditions of use ; in other words, it alone has the clinic on its doorstep.

Secondly, there is a practice (almost confined to Great Britain) whereby British Railways design and build, as well as maintain, their locomotives and rolling stock. These activities have, in fact, involved the formation of a vast engineering industry, and clearly justify its own facilities for research.

Thirdly, there is the more general, but no less cogent, argument that the existence of an internal research organization is in itself one of the best ways of ensuring that the results of research undertaken elsewhere are not ignored. It ensures a receptive contact with governmental, university, and industrial research, thus providing an insurance against isolationism in technical matters ; it also forms the official link with research on the Continental railways—through the research organization of the International Union of Railways, and on those of North America—through the Association of American Railroads.

Research and Development

In many self-contained industries it is desirable and practicable to form a group or department responsible for both research and development. This procedure has some advantages, in that it avoids the problem of defining what is research and what is development, and facilitates the rapid development of the outcome of research. It may, however, possess the drawback that research is sacrificed to development, and that research staffs, whose outlook and capabilities differ from those of development engineers, are not so free to operate to the best advantage in their appropriate field.

A decision on this matter depends partly on the type of research and the type of development in question. The combination may be most appropriate where research is intended to result in a flow of novel inventions which must be quickly embodied in new products for sale. In the case of the railways, where the outcome of research is not usually of this character, and where development cannot readily be carried out in, so to speak, an adjacent building, there are strong arguments for separating the functions of research, which can to a great extent be centralized in and from laboratory centres, and those of development, which require a different type of personnel and facilities which can only be found on the full scale within the other Departments of the Railway Organization. This conclusion, however, involves at least a notional and generally accepted distinction between research and development, as well as a willingness of both parties to co-operate when necessary and not to take

immediate umbrage if, as must happen occasionally, the rather ill-defined boundary is overstepped.

Legitimate Field for Research in Railways

What, then, are the generally accepted principles—for there is no written charter—within the Railway Executive as to the proper functions of its research organization? My own view is that these are to act as a fact-finding organization in the widest sense, as a consultant, sometimes as a referee, and above all as a critic and a stimulant.

As a fact-finding organization, the members of any research organization must be experts in the technique of instrumentation and measurement, upon which any form of scientific enquiry depends. In railway research there is also much scope for the adaptation of laboratory techniques to field work, since much research must inevitably be conducted on the track and in moving vehicles. As regards instrumentation, in addition to the development of instruments for research work itself, there is the further task of devising instrumentations for eventual routine use by other Departments. What I have described as fact-finding may be nothing more than a few measurements made in the course of a valid experiment, or it may be a basic research extending over a considerable period of time. In either case it provides data on which decisions can be made with greater confidence than when they are based upon personal opinion.

As a consultant, the Research Department can legitimately be looked to for assessments of new developments in science, and for expert advice on the properties of materials of all kinds; to act in this capacity demands the establishment of close contact with outside sources of research and information. This type of service acts as a useful check on the not always disinterested advice to which the Departments are exposed by technical salesmanship.

It is also common practice to call in the Research Department as a referee in reporting upon the respective merits of varying practices, or in investigating a problem or even a mishap in which more than one Department is concerned. From its impartial position it is able to measure, record, and analyse from a completely unbiased standpoint.

As critic and stimulant, the task of the Research Department naturally demands no small degree of tact, but the status of the Research Department, as described in the next part of this Lecture, shows that its position in the organization permits it to enter this delicate field.

The Railway Executive Research Department

Historical

It will be well known that there was no unanimity in practice on the former main-line companies, or among railway administrations abroad, as

to the nature, organization, or status of research departments, even if they existed at all. The unification of the railways of Great Britain on the 1st January, 1948, afforded a unique opportunity to consider the whole problem afresh, and since the Transport Act of 1947 specifically enjoined the British Transport Commission to see that proper provision was made, *inter alia*, for research, that body at an early date in its existence set up a committee, consisting of Sir William Stanier, as chairman, together with Sir Thomas Merton, Sir Charles Goodeve, Sir Henry Grey, and Mr R. A. Riddles to "examine and report on the arrangements for conducting research in the Railways and other undertakings vested in the British Transport Commission, and to make suggestions for the organisation of such work in the future." This Committee reported in November 1948, and so far as the Railways were concerned, recommended that the research facilities within the Railway Executive should be integrated into a single department under the control of a Director of Research responsible directly to the Chairman of the Executive. Contact and co-ordination with similar establishments existing or projected in other Executives was to be maintained through a Research Co-ordination Committee, under the chairmanship of a Chief Research Officer at the Commission, and of which the Director of Research of the Railway Executive was to be a member. The recommendations of the Stanier Committee were accepted by the Commission: the Railway Executive appointed a Director of Research, and eventually the new Railway Executive Research Department took shape and started to function on the 1st January, 1951.

It is perhaps desirable at this stage to emphasize that, although designated a Research Department, the organization we are considering would be more correctly termed the Railway Executive Scientific Service. It is responsible broadly for all activities that require to be carried out by scientifically trained staff with the facilities available in laboratories. About half of the total effort available is devoted to what can properly be termed research or work of a research standard. The rest is expended on analysis and testing, and on work of a day-to-day character.

The Problem of Organization

The precise relationship of a research department with the other units of an industrial organization is always a difficult, and indeed often a delicate, matter to decide. In the case of British Railways, the problem was further complicated since it arose at a time when a new organization of the Railways was being developed, and by the need to serve not only the Railway Executive Headquarters but also the six Regions of British Railways, each of which has its own team of railway officers under a Chief Regional Officer. Geography and the location of such laboratories as existed also added to the difficulties in deciding upon the best organization.

The decision that there should be a single Research Department,

rather than an organization (horizontally) in each Region, or (vertically) for each Department or Function, was no doubt dictated largely by what was immediately practicable, and by the desirability of having one effective organization rather than several small units. On the other hand, it is becoming increasingly clear that the mere integration of rather limited existing facilities cannot produce an organization capable of meeting the needs of both the Railway Executive itself and the Regions, some of which had not hitherto had access to a service of the type now proposed. Although there has been a small increase in accommodation available during the past year, and some additional staff has been authorized, the times have not yet been favourable to expansion.

Nevertheless, the general conception of the new Research Organization has not been seriously questioned, and the main criticism has been that problems put to the Department often take a long time to solve.

This is mainly owing to the fact that the organization is not yet large enough to undertake the growing volume of work, and also because senior officers of the Department have initially had to devote much time, often as chairmen of committees, in advising on questions of the standardization of methods and materials (from which large economies are now flowing). The fact that they were called upon to do so, whilst slowing down the prosecution of research, is evidence of the extent to which the Department has from the start been accepted as a consultant.

The position of the Research Department within the Railway Executive is somewhat analogous to one of the Research Associations in relation to the Industry it serves and by which it is largely financed. The various Departments both at Headquarters and in the Regions correspond to the member firms of the Association; both carry out most of their own development work within their own premises, but they remit to the Research Department or to the Research Association problems of a more general and scientific character, for which they cannot individually provide the specialized research staff and facilities, and they look to their common research centre for advice and guidance on subjects rather beyond the field of their own day-to-day activities.

The Present Organization

In drawing up an organization which would "integrate the existing research facilities" of the former companies, the fact had to be faced that only one of these former companies—the London, Midland and Scottish—possessed a Research Department of the type now contemplated by the Stanier Committee and approved by the Commission and the Executive. Most of the others possessed chemical and metallurgical laboratories attached to and mainly concerned with the work of the Chief Mechanical Engineers' Departments. Work of a research character in metallurgy or engineering tended to be conducted sporadically and was largely dependent upon the personal outlook and inclination of individual engineers. How-

ever, whatever was readily identifiable as a laboratory or research unit was incorporated in the new organization, which in itself was inevitably based upon the former L.M.S. Research Department.

The Director of Research, with a small personal staff, is located in London at the Railway Executive Headquarters. Such an arrangement is helpful, and probably essential, from the point of view of maintaining contacts with the other Departments at the highest level, and in providing some visible evidence at headquarters of the existence of the Research Department.

The Department is then divided into six major divisions, each under the control of a Superintendent. The Superintendent of the Chemistry Division is also located in London, and controls laboratories at Glasgow, Horwich, Darlington, Crewe, Derby, Doncaster, Stonebridge Park, Wimbledon, Stratford, Ashford, and Swindon. At Derby there is both a chemical laboratory and also a laboratory dealing with paints, protective coatings, and corrosion. The other Divisions have their headquarters at Derby (Engineering, Metallurgy, Physics, and Textiles) and London (Operational Research).

The Director of Research, his Deputy, and the Superintendents meet regularly as a Research Superintendents' Committee which reports to the Railway Executive, and, as previously mentioned, the Director of Research is himself directly responsible to the Chairman. The scientific view can thus be expressed at the highest level when occasion arises, but the great majority of the Department's problems is initiated by individual Railway Executive Members, or by Officers in the Regions, and reports are made direct to these Members and Officers. This practice establishes a direct relationship with each client, analogous to that in the legal or medical world, a matter of importance in developing and maintaining confidence in the Research Department.

Apart from the Divisions, and directly responsible to the Director of Research, is a Librarian located in the main research centre at Derby with a subsidiary branch in London. The functions of this unit are those of the typical special library, and it serves both the research staff and officers in other Departments. It publishes a monthly review of technical literature, aimed mainly at bringing to the notice of the Departments research reports and relevant articles in the less frequently consulted English and foreign technical periodicals.

The present complement of staff is 319, of whom 221 are scientific and technical staff, and of these 80 possess professional or academic qualifications in their appropriate subjects. Unfortunately, this staff is housed in separate premises at thirteen different centres. It is hoped in the near future to close the three small chemical laboratories in the London area and to incorporate the chemical work arising in this area, as well as certain types of chemical research proper, in a central laboratory in or near London. Even then, the inescapable facts of geography, which are

inseparable from a railway system, prevent a neat and easily supervised research establishment, and increase the difficulty of staffing some of the more isolated laboratories.

Of the existing laboratories, only one building, fortunately the most important, can be described as reasonably good by modern standards. This is the London Road Laboratory at Derby, which houses metallurgy, engineering, and protective coatings. In recent months an additional building has been taken over, also in Derby, and converted into quite good laboratories for the Physics and Textiles Divisions. A chart of the organization is given in *Fig. 1*.

I have dealt in general terms with the functions of the Research Department of the Railway Executive, and have outlined the history of its development. I should now like to illustrate these functions by giving a few examples of the work of each of the Divisions.

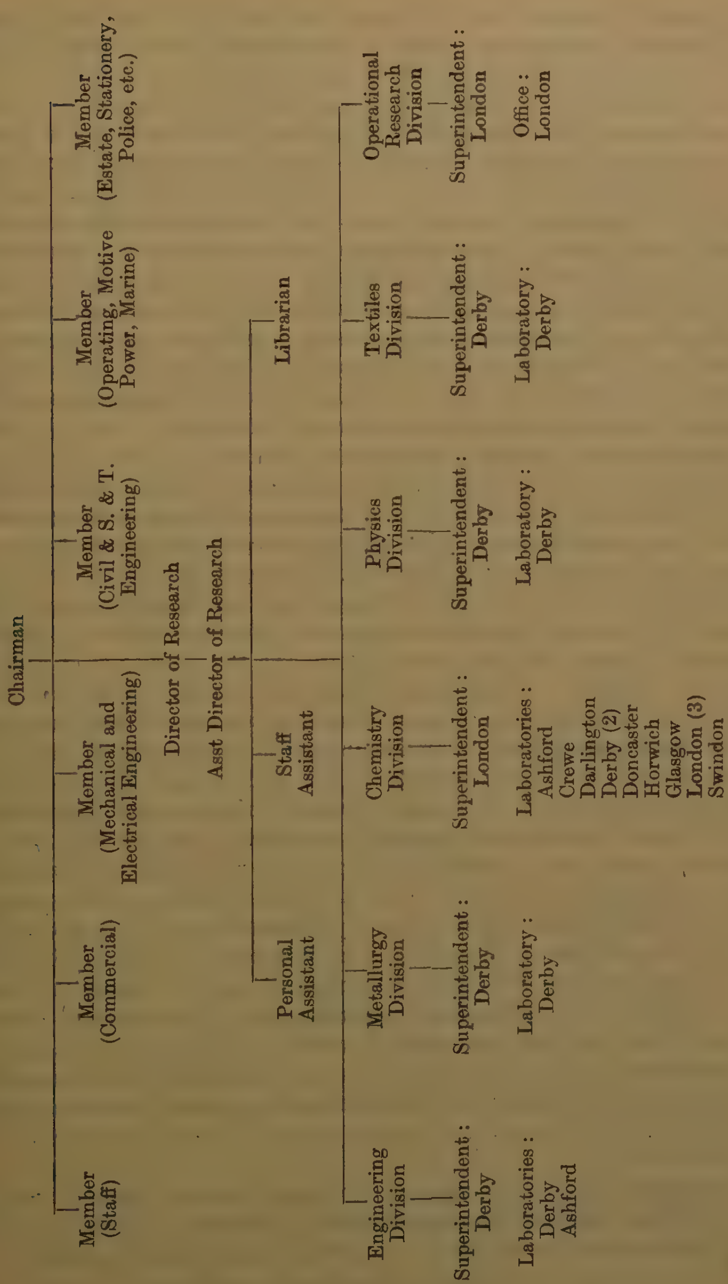
Chemistry Division.—The Chemistry Division is numerically the largest, since it is required to give day-to-day service throughout the regions on an exceptionally wide variety of subjects. A considerable amount of this work relates to the examination of materials and products, either purchased from industry or manufactured in railway premises. Though not research, this work often points the need for research, and aids in the development of experts in particular materials who may later be able to handle research in those fields. Further, the knowledge thus accumulated is effective in reinforcing and safeguarding the position of the engineer and the purchasing officer in their dealings with the outside manufacturer. This has been particularly true and advantageous in the case of paints, oils, and textiles, where familiarity with both the industry and the special railway requirements has been helpful in interpreting the needs of the latter to the manufacturer.

Other duties of an almost statutory nature which fall to the Chemistry Division consist in giving advice on the classification of new traffics for charging purposes, on the conditions of carriage of dangerous goods, and in assessing the validity and legitimate extent of claims made against the railways for damage to goods during transit. Many cases of fraudulent claims have been exposed as the result of investigations carried out in the chemical laboratory.

The technical problems of a railway are of course primarily of an engineering character, and the scope for long-term chemical research is more limited. The principal studies tend to be related to fuel, lubricants, waters, building materials, corrosion, and protective coatings.

As regards fuel, the annual locomotive coal bill of the Railways is of the order of £42 million, so that a small improvement in the method of using it could result in substantial economy. The study of the economical use of locomotive coal is one at least as much for operational research as for chemistry, bearing in mind, for example, that probably not more than 60 per cent of the total amount of coal is directly used in hauling trains.

Fig. 1



The Research Department of the Railway Executive

On the purely chemical side, however, important work has been carried out in studying combustion efficiency in relation to firing methods, and to the instrumentation necessary in measuring this in the course of locomotive testing.[1, 2]

In the case of lubricants, the oils required by locomotives, carriages, and wagons do not present the more critical problems associated with internal combustion engines or gears. Very few cases of failure can, in fact, be attributed to the quality of the oil; most of them are the result of shortcomings in design or maintenance such as overloading, interruption of the oil supply, or the presence of dirt in the oil. In times of material shortages, such as were experienced in the 1939–45 war, the chemical problem arises of suggesting and assessing possible alternatives such as for rape oil in the manufacture of compounded oils; at the present time attention is being directed to the suitability of colloidal graphite and even to the interesting properties of molybdenum disulphide.

An example of an investigation which might lead to results far beyond its original scope is a recent attempt to formulate an improved grease for wagon axle-boxes. This assumed anachronism is of course disappearing, and no new stock is being built with grease-lubricated axle bearings. Nevertheless, a sufficient number of such wagons still remains to be a source of inconvenience in marshalling yards and in the occurrence of hot boxes. In a grease-lubricated axle-box, the grease reservoir is situated above the journal, and the grease flows on to the journal as it softens under the influence of heat generated in the bearing. This primitive action is dependent upon a grease having closely controlled melting-point characteristics, such as are obtainable in emulsified mixtures of water and soda soaps, mineral oil, tallow, palm oil, or wool grease. The traditional grease may contain up to 50 per cent of water, which in service is gradually lost by evaporation, thus changing the properties of the mixture, raising its melting point and rendering it inoperative as a lubricant until the bearing temperature has risen to an undesirable degree. Cold weather naturally intensifies the trouble. Clearly the aim was to produce a waterless grease with an equally narrow melting-point range, considerable stability, and resistance to frost. These conditions are substantially met by the addition of an appropriate thickening agent to the standard wagon-axle oil. The work has now reached the stage of practical trial, and it has been mentioned at some length as a good example of the scientific re-examination of a traditional and obsolescent practice which may solve an existing difficulty and might even prove a real alternative to oil lubrication.[3, 4]

The suitability of water supplies, both for drinking purposes and for locomotives, is a matter coming within the jurisdiction of the chemist, and in the case of the former, of course, both chemical and bacteriological

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[1] The references are given on p. 240.

examination is called for. The chemical control of water softening for locomotive purposes, which is work of a routine character, is at present under review and it is likely that it will be made the responsibility of the Department which uses the treated water. However, problems do arise in the field of water treatment, and much effort has been devoted within the past few years to the development of a suitable anti-priming agent to reduce the amount of blow-down necessary. A particular study has been made of suitable polyamides and their preparation in a form suitable for introduction into the feed water. The material eventually developed has proved successful in enabling the locomotive boiler to carry a concentration of 2,000 grams per gallon of soluble salts without causing priming, whereas about 200 grams per gallon is the normal limit without any foam-inhibiting addition.

In the original organization of the Department as set up in 1951, provision was made for a separate division concerned with protective coatings and corrosion work. This was done partly because the subject is not exclusively chemical in character, but it soon became clear that all the work dealing with the subject would not be centralized, since much of the work and the day-to-day advice required was of local origin. Since July 1952, therefore, this division was amalgamated with the main Chemistry Division, and a good deal of the day-to-day examination of paints and allied materials is performed in some of the Area Chemical Laboratories. Research on protection, cleaning, etc., and on corrosion matters is, however, concentrated in the specialized laboratory at Derby.

The importance of the correct choice of protective coatings, methods of surface preparation and of application, as well as methods of cleaning for railway equipment cannot be over-emphasized. Full protection of the underlying metal or wood, the lengthening of periods between successive re-painting, and the use of cleaners which are effective and yet not harmful to the paint or the metal, are questions which can lead to financial savings quite out of proportion to the cost of the materials and the cost of their application.

To be effective, research in this field must be based first upon the sympathy of the engineer and secondly upon a full knowledge of the conditions of both application and subsequent service. When British Railways were formed in 1948, each constituent Railway possessed varying specifications and methods which it was clearly right to standardize. This was done through the agency of committees both on the mechanical and civil engineering sides, and in both cases the Superintendent of the Chemistry Division acted as chairman.

The painting of locomotives and, in particular, of carriages is complicated by the facts that, in addition to preserving the underlying metal from the ravages of corrosion, the maintenance of decorative effect or smartness is of considerable importance. Provision must be made therefore both for designs and for types of material that lend themselves to

frequent cleaning, and the cleaning process in turn must be made as easy and effective as possible and it must have the least possible injurious effect on the paintwork or on any bare metal with which it may come into contact. Further, the nature of the dirt and grime to be removed requires particular consideration, since it is much more troublesome than in the case of, say, a car or a lorry. In tunnels particularly the railway vehicle is subjected to the effects of a very highly polluted environment, whilst on the locomotive and also on the carriages the effect of oil splashes is a factor which seriously complicates the problem. In viewing all these factors as parts of a whole, the paint technologist has a great field for the exercise of his specialized knowledge and ability. Not only do problems for laboratory and field research emerge at each stage, but there is also required the continual insistence and education on proper methods of application and after-care, and through the medium of the committees already mentioned this research and near-research work has been embodied in specifications (mainly of a performance character), schedules, and codes of practice.

The painting of rolling stock has been mentioned at some length, because it involves other associated problems of interest, but the protection of structures and buildings is in itself of equal importance, and presents the added difficulty that the actual painting process cannot be carried out under shop conditions. The subject was, however, fully dealt with in Papers presented to this Institution.[5, 6]

Before concluding this reference to protective coatings, however, mention should also be made of the development of a self-cleaning white paint for application to surfaces which act as markers, for example, signal posts; odourless paints for use in hotels; interior finishes with high resistance to flame spread; non-inflammable paint strippers, etc.

Engineering Division.—The technical activities of railways are essentially of an engineering character, and the responsibility for design, maintenance, and development rests with the Engineering Departments. The Engineering Division in the Research Department, however, provides assistance in any or all of these fields by providing basic design data, and by measuring (or providing the means of measuring) the performance of existing or proposed structures and units, or of details or of materials.[7]

By the provision of basic design data is meant, for example, data on the fatigue properties of materials or components; stress distributions in a wide variety of components ranging from bolts and rails to bridge members and locomotive frames. Performance measurement, on the other hand, covers such studies as the behaviour under laboratory conditions of a carriage-door lock or a rail spike, or field investigations on the lateral resistance of ballast or the movement[8] and forces between rail and tire. In both classes of problem, the emphasis is on measurement, which in turn involves instrumentation. Much of the laboratory work can, of course, be performed on standard equipment whether this be fatigue

machines, high speed cine-cameras, the photo-elastic bench, or the wind tunnel.[9] But in many cases, particularly for field work, there is the need to develop specialized measuring equipment. This may in turn range from quite small items, such as pressure cells for the measurement of pressure distribution in ballast and formation, to the mobile testing plant [10] for conducting constant-speed tests for locomotives, or for the measurement of train resistance.

The consideration of the relative advantages of different forms of motive power, and the design of locomotives of whatever type, are not matters falling within the province of the Research Department. The Engineering Division does, however, participate in the testing of complete locomotives through representation on the Locomotive Testing Committee, which controls the work carried out on the stationary plants at Swindon and Rugby, with the Mobile Testing Plant (originally developed in the Engineering Division) and with Dynamométer cars. It can thus play an important part in the planning of this work, in the interpretation of the results, and in the development of instrumentation. One interesting example of such instrumentation is the conception of a weighing grate [11] for use in the mobile tests. The interpretation of the results of such tests is a subject to which my earlier reference to one of Unwin's addresses particularly applies. It is in a matter of this kind that research staffs have an advantage, not because they necessarily bring abler minds to bear on the problem, but because they have, by the nature of their duties, more time to give to the study and critical analysis of the very large amount of data produced in the course of tests of this kind.

More particularly in the province of the Research Department within the locomotive field are such matters as the behaviour of the locomotive on the track,[12, 13, 14] for example, the measurement of flange and lateral forces set up during motion, the change of weight distribution between the various axles and wheels during running, the performance of springs, air flow problems including smoke lifting, etc., all matters in which accurate data can only be obtained through the use of somewhat specialized instrumentation. Again, the defects which arise during service on locomotive frames, involving expensive maintenance, is a subject capable of being studied in the laboratory by means of a section of an actual frame that can be appropriately stressed, by models on which questions of rigidity can be studied, and by photo-elastic and mathematical investigations.

Again, in the case of carriages and wagons, overall design is the function of the Mechanical Engineering Department, but questions of resistance to motion have been handled by the Research Department, and in connexion with wagons the Engineering Division was entrusted by the Executive with the overall planning of an investigation to study the behaviour of continuous brakes on freight trains, involving a comparison between the respective merits of the vacuum and air brakes. The possible

application of continuous brakes on freight stock offers great potential improvement and economy in operating, not primarily because it would enable the speed of freight trains to be increased, but because by bringing the speed of freight trains closer to that of passenger trains a notable increase in line capacity and reduction in delays could result. Continuous brakes are of course almost universal outside Great Britain, but the problem here is rendered more difficult by the fact that centre automatic or even screw couplings are not fitted to freight stock in general, and the behaviour of the continuous brake on long heavy freight trains with loose couplings has necessitated a series of most carefully planned experiments.

So far as buildings are concerned, engineering research has been concerned mainly with wind-tunnel studies on wind pressures on platform awning roofs, and on various problems of smoke clearance from stations, cuttings, tunnels, and above all, engine-shed roofs. This latter is in many cases a most difficult problem to which no general solution may be possible owing to the very varied situations in which these sheds are necessarily placed.

Within the past few years there has been a noticeably increased demand for precise information as to the actual stresses induced in bridge members, a particular case being that of wrought-iron bridges which have been in service for perhaps a hundred years or so, and which were clearly not designed with a view to present-day traffic and loading. The problem falls into two parts—the measurement of stresses in members suspected to be overloaded, and a study of the deterioration of wrought iron with particular reference to a possible diminution in the resistance to fatigue. This latter work is, of course, performed in the laboratory on samples taken from actual bridges, but the stress determinations are made on actual sites with the help of a departmental vehicle carrying the necessary recording gear associated with the strain gauges attached to the bridge members.

Perhaps the largest single group of engineering investigations during the past five years has been that associated with some aspect of the permanent way.[15, 16, 17]

Commencing with the formation, investigations have been started—in conjunction with the Civil Engineering Department and with the Road Research Laboratory—in the determination of pressure distribution in the formation, and it is hoped to continue this in relation to the depth of ballast. This is a subject in which there are wide variations in practice between different countries, and a controlled test length in which the depth of ballast ranges from 6 to 24 inches has been installed, and records of behaviour and maintenance are made at intervals. Whilst the subject of soil mechanics is one that has attracted many investigators in recent years, much less is known about the behaviour of a bed of stones. Another aspect of ballasting is the lateral resistance offered by the ballast to the sleepers, which depends upon the type of material, the depth and degree

of consolidation, and the size and slope of the shoulders. The contribution made by the shoulders may be quite critical when the sleepers are lifted off the ballast ahead of the locomotive, between succeeding axles, or at the rear of the train. Lateral resistance measurements, with various shoulder sizes in granite slag and ash, have produced factual data upon which new British Railway standards have been based .

As regards sleepers, the Engineering Division has been closely associated with the development of concrete-sleeper design [18] and with the method of attachment of the rails thereto, both from the point of view of protecting the concrete from galling, and with regard to the possible provision of insulation when track circuiting is employed. With the introduction of the new flat-bottom track, the question of fastenings demanded considerable study, and the Division has contributed materially to knowledge of the behaviour and design of spikes.

Finally, with the rails themselves, much data on the service conditions to which rails are subjected has been obtained by stress measurements—in fishplates also—and by photo-elastic and mathematical studies. This was fully used when the new flat-bottom-rail profiles were finally laid down. Rails fail primarily by fatigue, and equipment has been devised whereby fatigue tests can be carried out on the full-scale rail,[19] thus enabling an assessment to be made of the progressive deterioration of a rail caused by loss of section arising from wear and corrosion and by reduction in fatigue strength consequent upon surface deterioration. This information has made it possible to offer a more logical basis for predicting rail life.

Metallurgy.—The Metallurgy Division is concerned purely with investigational and research work in metallurgy, apart from operating a mobile radiology service. It is not concerned with day-to-day metallurgical control in the Executive's workshops, this function being performed under the guidance of Regional Works Metallurgists responsible to the Mechanical and Electrical Engineers in each Region. These Regional Works Metallurgists do, however, meet regularly with the appropriate Research Department Officers as a Joint Works Metallurgical Committee. There is thus a regular channel through which metallurgical problems arising in the workshops can reach the Research Department, and through which the results of research can in turn be made known to those in a position to introduce them in practice.

The investigational work performed in the Metallurgy Division often arises from failures in service of metal components of locomotives, rolling stock, and track. Many of these are of a recurring character, and in the case of rails, for example, they are classified locally in accordance with categories laid down in the Railway Executive Rail Failures Handbook, which was produced in 1948 by a joint committee of metallurgists and Civil Engineers' representatives. Only unusual rail failures are now sent to the Division for examination, but statistical data regarding the nature

and occurrence of these defects has, and still is, being accumulated, the study of which can lead to important conclusions. Data from a particular Region showed the predominance of fatigue failure, the overwhelming tendency for these failures to occur on the right-hand rail, and the probable influence of a relatively small number of locomotives with driving-axle loads somewhat higher than normal.

Regarding rolling stock, the study of the corresponding statistical data has not so far been attempted except in the case of wagon drawbar hooks, since in the majority of other details liable to recurrent failure, there are still so many variations in design which would make the statistical study of their behaviour much more difficult. The number of individual investigations into the failure of locomotive and rolling-stock components is still considerable, although it is becoming better realized that most failures are not so much caused by faulty material as by matters of design and maintenance.

Although not strictly a matter of research, a metallurgical advance of great practical and economic importance since the nationalization of the Railways has been the reduction of the wide variety of non-ferrous alloys to five copper-base alloys and four white metals. This has been done by a committee of engineers and metallurgists under the chairmanship of Mr T. H. Turner, the Superintendent of the Metallurgy Division.

The Division has paid much attention to the subject of rail wear, and at the outset felt the need for an accurate method of measuring wear so that the time neecssary to measure the wear of rails in service should not be unduly prolonged. This need led to the production, in collaboration with an outside instrumentalist, of a sensitive contorograph which would record the profile of the rail head with a magnification of 10. At the same time, the mere recording of wear was not in itself helpful without a knowledge of the amount of traffic passing over the particular rail being studied. This information can be obtained somewhat tediously by inspection of the signalman's trains book, but it is now much more conveniently recorded by means of a wheel-counting device developed in the Engineering Division of the Department.

Special attention has also been paid to the combined—or rather the alternate—effects of wear and corrosion in producing loss of metal from the rail head. Laboratory tests on an otherwise appropriate quarter-scale machine consistently failed to reproduce the rate of wear of rails and tires met in service, and the cause of this discrepancy was thought to lie mainly in the absence of periods of atmospheric corrosion. It was thought not unreasonable to suspect that for most of the year in Great Britain, oxide layers could form on the rail head between the passage of successive trains, and that the periodic removal of this layer, in addition to normal wear, might greatly affect the overall rate of loss of metal, since the corrosion would always be occurring at the initial corrosion rate whilst the actual wear would be increased on a corroded surface and per-

haps also increased by the abrasive action of the products of corrosion.[20] Corrosion studies are always slow and sometimes indeterminate, but the work so far done, both in the laboratory and on the track, certainly appears to be confirming this hypothesis.

The mobile radiology service to which reference has already been made is at present maintained by a Phillips 150-kilovolt-ampere set installed in a vehicle complete with dark room and staff accommodation. This vehicle can be taken into any works at which static equipment is not available, or it can be taken out on to the line for the radiography of bridges and other structures. A recent interesting extension of this work is the use of a cobalt 60 source which has proved very convenient and successful for the location of reinforcements in concrete. A certain design of reinforced-concrete underbridge incorporates prestressing cables contained in steel tubes, and in the course of manufacture it was discovered that these could undergo considerable displacement. It was therefore desired to ascertain to what extent such displacement, if any, had occurred in the case of a unit already in service in a main line. The technique employing the cobalt 60 source was found to be a most convenient method of obtaining the necessary information, which in this particular case fortunately proved to be reassuring.

Operational Research.—The Operational Research Division provides the means of studying non-technical problems from an independent and impartial angle, and with the aid of various mathematical and statistical techniques which have been developed for the purpose. It involves the scientific planning of fact-finding investigations, the analysis of observed data, and in some cases the devising of new methods of assessment. The term operational is intended to cover any activity, method, or function, other than one of technical process or design; and it is not of course restricted to operating in the specialized railway sense.

Operational research, under that name, came into being during the war when it was realized that the particular outlook and training of scientists could be of value to the Services, in assessing the relative value of particular weapons or defences in relation to the effect involved or to the precise method of using them.

Studies of the same kind were not unknown in industry long before the war—even on the railways—since effective management must be presumed to be based upon the best use of an organization and its physical equipment. Nevertheless, the specific introduction of the scientific approach, the scientific planning of observations and experiments, and the use of mathematical and statistical aids were a new feature, and the Stanier Report laid considerable emphasis on the introduction of this new branch of applied research to problems arising before the Commission and its Executives.

It seemed at the outset that a particular field of usefulness for operational research lay in the re-examination of traditional practices which,

although they had been continually reviewed and developed, had perhaps not been fundamentally and critically examined since their inception. Opinions and practices can so readily be retained long after the circumstances which originally gave rise to them have changed fundamentally. In the light of this reasoning, therefore, the Division has studied such subjects as the fundamental principles upon which to base a bonus system for Goods Station staff, the most logical and effective quarter on which to place responsibility for the checking of stores issued to stations, and the present-day conditions and physical provisions for the conveyance of passengers' luggage, having regard to changes in social habits that have occurred since the nineteenth century.

The Division has also participated in an enquiry, which has many facets, into the actual use made of locomotive coal. It has been suggested that the proportion of coal actually used in hauling trains is not more than 60 per cent of the total consumed by locomotives, and whilst it may be easy to visualize what becomes of the remainder, a more careful analysis of this unproductive combustion might well point the way to economy.

The Operational Research Division is also the natural link with developments in applied physiology and psychology which might help to solve certain human problems in the railway industry, and it also keeps in touch with such new fields as are covered by ergonometry and similar composite sciences.

Physics.—The purpose of forming a Physics Division [21] was partly to cater for the wide variety of miscellaneous problems involving classical physics—heat, light, sound, optics—and also to provide a small team of what may be termed general scientists, who could examine rather ill-defined problems and explore new techniques which do not clearly fall within the province of the other divisions. To this has been added a small mathematical section which carries out mathematical and statistical studies, mainly those arising in the course of technical investigations in other branches of the Department.

In the realm of heat, continuous attention has been given to problems of heat transfer, insulation, and refrigeration arising in connexion with the conveyance of traffic requiring controlled temperature conditions.[22] Here the Physics Division has contributed to the performance of insulated stock in the study of heat flow through the walls of containers, the assessment of the relative merits of different insulating materials, and the optimum utilization of refrigerants. It has also provided a valuable link between the work of the Food Investigation Board of the Department of Scientific and Industrial Research and the Commercial Department of the Railways, and has provided the scientific observation necessary in recording the behaviour of foodstuffs in transit, especially in cases where controlled temperature or ventilation is all-important. With the more recent introduction of heavily insulated vehicles, some of the simplifying assumptions hitherto adopted in designing the insulation and calculating the required

amount of refrigerant were no longer justified. The mathematics of problems involving variable heat flow can be very complex, but relaxation methods [23] have been employed with considerable success to the difficult problem in which the vehicle starts warm and is then loaded together with the refrigerant which is required to exercise its control within quite a limited time. Again, special problems of heating and ventilation arise in connexion with passenger stock and buildings.

The ordinary run of lighting problems do not fall within the province of this Division, but an interesting case occurred a few years ago when it was desired to try and improve the visibility of the actual water level in a certain type of locomotive water-level gauge. This particular form of gauge employs a reflecting block of glass, and the indication of level is brought about by the difference in reflexion from the glass/water interface on the one hand and the glass/steam interface on the other. The original type of glass block was corrugated, and when viewed normally, total reflexion should have occurred when there was steam behind it and near blackness when there was water. However, the erosion of the internal corrugations tended to blur this distinction irrespective of any external illumination, and a solution of the difficulty was achieved by the use of a block carefully designed with reference to the brightness of the reflexions at different angles, having a plane face in contact with the water and steam, and which remained reasonably plane in spite of erosion by the boiler water.

Experience with the behaviour of insulation spaces and insulation materials has focused attention on the adverse influence of moisture in such spaces, both from the point of view of deterioration in the insulation itself and in respect of the promotion of corrosion. In the study of this problem in the particular case of a prefabricated wall construction, a method of following the humidity changes was devised. This was based upon the accidental observation that if light is directed approximately axially along a glass thermometer stem, the light is largely returned by total internal reflexion at the rounded tip of the bulb, whereas if spots of moisture are deposited thereon, they show up as bright pin-points on a darker background. A small ether jacket was made for the bulb, and the whole mounted in an aperture in the wall with a small window and mirror arrangement whereby the bulb tip could be viewed from outside.

Another interesting example of instrumentation arose in the study of the electrical insulation of sleepers. The conventional method would involve the removal of the rails, and the lifting and measurement of resistance of each sleeper in turn. This tedious and inconvenient method was eventually avoided by the application of an A.C. potential across the rails, after which, by means of a portable pick-up, the current flowing through each individual sleeper could be readily indicated. In this way it is quickly possible to pick out particularly bad sleepers in a length of track-circuited line.

Textiles.—The Textiles Division was established in order to make available testing, research, and advisory facilities on an important group of materials whose properties and behaviour are not normally familiar to railwaymen, technical or otherwise. The need for expert guidance in the choice of textile materials, particularly in view of the newer synthetic fibres now coming on to the market, will be apparent in view of the large annual expenditure on textiles of all kinds, amounting to £10 million a year. The variety of requirements is surprisingly great, ranging from uniforms, ropes, upholstery, carpets, and canvas for wagon and cartage sheets, to a multitude of smaller items such as towels, dusters, threads, and twine. In performing the services required in connexion with the selection, testing, and performance of textiles, the Division is fortunate in the assistance available to it from the Research Associations concerned with cotton, linen, and wool, and laundering.

Since the Railways are users and not manufacturers of textile products, the opportunity for research leading to new products does not exist, though physical and operational research on the performance of such items as ropes, upholstery materials, carpets, lubricating pads, etc., has enabled the specific requirements of railway textiles to be interpreted to manufacturers in a way that would not be possible in the absence of the link formed by the Textiles Division.

In conclusion, I would emphasize that my purpose has been to paint a broad picture, rather than to give details of the outcome of particular investigations. These are the province of the individual investigator when presenting the results of his own work, and I should not wish to encroach upon the field of my colleagues in the Department.

I have tried to indicate the practical and economic advantages of having brought together the scientific resources of the Railways—both in manpower and equipment—into a single organization, which, although still growing, is large enough to afford reasonable premises and equipment, and to attract men of the right calibre. It is to be hoped that the future will enable this expansion to continue, and that no attempt will be made to disperse the research effort of British Railways among smaller and less effective units.

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Mr W. K. Wallace, in moving a vote of thanks to the Lecturer, said that he had been a long-standing and satisfied customer of the Research Department since it had been formed originally by the L.M.S. Railway.

The setting up of the department had been considerably criticized by some of the older officers and others who had been proud to have chemists of their own lurking in some corner of their shops, and who maintained that the new department could not do anything that their own men could not do. It had been a great advantage, however, because it had been possible to get all branches of research carried out by referring problems to the department, whereas if one were given a research staff on a departmental basis it would not be possible to have the field covered so adequately.

Mr Wallace, as a civil engineer, would want some jobs done purely by engineering research. There were others, however, which would require metallurgical research, and a Civil Engineering Department would never have been able to afford the services of a good metallurgist; one could not have made the job attractive enough to get a good man and get the salary passed by the Directors. By concentrating the research workers in a separate department it was possible to make the best if not of both worlds at any rate of the present world. It was very likely the fact that there had been a comprehensive organization in existence on the L.M.S. Railway that had led British Railways to continue it, although Mr Wallace felt sure that they must have been advised by some of their officers that it would be better to departmentalize the work.

A great deal of the success of the "bedding down" of the Research Department had been due to the personality and tact of Mr Herbert. Some of Mr Herbert's staff had been inclined to be omniscient in their early days, but he had never permitted that error. To the District Engineer who had run his district for ten or twenty years, it was *his* district, and he was hardly willing to welcome the Chief Engineer, much less anybody else. If someone came into the district and said that he wanted to test the reactions of sleepers, it was liable to cause the District Engineer to write an irate letter of protest to headquarters, pointing out that if they had lost all confidence in him he should be told, and not find someone coming from outside to do things that he had known about for years.

Mr Wallace noticed that some of the jobs mentioned in the Lecture had been jobs initiated many years previously and which were evidently still with them. He hoped that Mr Herbert would put his successor wise to them when the time came for him to retire.

Mr Wallace joined with Mr Herbert in hoping that the Research Department would be left in being and not cut up into small units.

It had been very interesting to hear how the Research Department was continuing, and the Institution was greatly indebted to the Lecturer for giving a very interesting account of work which was little known by the public at large or even by the engineering profession.

Professor A. J. S. Pippard, who seconded the motion, said that since the Lecture was an Unwin Memorial Lecture he might be excused for referring at the end of it to Unwin himself. Having occupied for twenty years half the Chair which Unwin had adequately and fully occupied for so long, Professor Pippard was still amazed at the breadth of Unwin's interests and the variety of the subjects in which he had been able to undertake research.

There could be no doubt that Unwin had contributed extensively to knowledge in a very wide field, from the study of steam engines to that of masonry dams, from machine design to the flow of, and abstraction from, the water of the Thames.

He would undoubtedly have been interested in the topic chosen for this Memorial Lecture and would have been delighted to play a part in the kind of research described. Whether he would have been quite so delighted to be mixed up in an elaborate organization for research was not so certain. Probably in his time research was viewed as essentially a personal and individual affair, and its elaborate organization only developed later. It was with great pleasure that Professor Pippard seconded the vote of thanks.

The vote of thanks was carried by acclamation.

ORDINARY MEETING

20 January, 1953

HENRY FRANCIS CRONIN, C.B.E., M.C., B.Sc. (Eng.), President, in the Chair

The President announced that the Council wished to recommend to the members present the election of the Right Honourable C. D. Howe as an Honorary Member of the Institution. The form of the Council's resolution was as follows :

> " That in recognition of his distinguished services to the Dominion of Canada, formerly as a civil engineer and subsequently as one of the leading statesmen, the Council recommend to the members present at this Meeting the election of the Rt Hon. Clarence Decatur Howe, P.C., LL.D., B.Sc., as an Honorary Member of this Institution."

Mr Howe had had a very distinguished career in Canada, first as an engineer and later as a member of the Canadian Government. He obtained an engineering degree at the Massachusetts Institute of Technology and had been Professor of Civil Engineering at the Dalhousie University, Halifax, Nova Scotia, from 1908 to 1913. He subsequently became one of the leading consulting engineers in Canada, practising in Ontario. In 1935 he had been appointed Minister of Railways and Canals and Minister of Marine in the Canadian Government. He had been Minister of Transport in 1936, Minister of Munitions and Supply in 1940, Minister of Reconstruction and Supply in 1946, and had been Minister of Trade and Commerce since 1948. He had also deputized for the Prime Minister during the latter's absence from Canada.

The Council's recommendation was agreed to unanimously.

The Council reported that they had recently transferred to the class of

Members

Robert Boog Watson Bannerman, B.Sc. (*Edin.*).
Cecil Vernon Berry, B.Sc. (Eng.) (*Lond.*).
Cyril Henry Bunclark, B.Sc. (Eng.) (*Lond.*).
Moore Hamilton Elliott, B.Sc. (Eng.) (*Lond.*)
Thomas Louis Farnes.
Geoffrey Dillworth Lloyd, M.A. (*Cantab.*).

and had admitted as

Graduates

Henry Russell Shaw Abrey, B.Sc. (*Natal*).
William Peter Alexander, Stud.I.C.E.
Robert Dudley Anchor, B.Sc. (*Birmingham*).
Noel Barber Anderson, B.Sc. (*Edin.*).
John Huie Andrew, B.Sc. (*Glas.*), Stud. I.C.E.
Geoffrey Ernest Bamford, B.Sc. (Eng.) (*Lond.*).
Maurice Albert Frederick Bartley, Stud.I.C.E.
Robert Derek Binns, B.Sc. (Eng.) (*Lond.*).
Leslie Spencer Blake, B.Sc. (Eng.) (*Lond.*).
Vincent James Blundell, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Robin James Bradbury, B.Sc. (*Bristol*),
Laurence Frank Bradley, B.Eng., (*Liverpool*).
Maurice George Brothwell, B.Sc. (Eng.) (*Lond.*).
David James Brown.
Derek Clifford Brunning, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Denis Buckley, B.E. (*National*).
Francis Hubert Bullock, B.A. (*Cantab.*).
Raymond Free Bunce.
John Bundred, M.A. (*Cantab.*), Stud. I.C.E.
Robert Burns, B.Sc. (*Belfast*), Stud.I.C.E.
Roy Brockwell Bussell, Stud.I.C.E.
Dennis Butterfield.
George Ramsay Carr, B.Sc. (Eng.) (*Lond.*).
Brian John Catterall, B.Eng. (*Liverpool*), Stud.I.C.E.
Peter Gerald Cavalier, B.Sc. (*Bristol*), Stud.I.C.E.
Richard Gordon Cecil, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
William David Champion, B.E. (*New Zealand*), Stud.I.C.E.
Alan Baskerville Chapman, B.Sc. (Eng.) *Lond.*).
Denys Burrell Charnock, B.Eng. (*Liverpool*), Stud.I.C.E.
Frank Eaglesfield Chasemore, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Reginald Clare, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Basil Edward George Clark.
Brian Thomas Clark, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Noel Frederick Clark, B.Sc.Tech. (*Manchester*).
James Dennis Clarke, B.Sc. (*Durham*).
Ronald John Clear, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Frank Cliff, B.Sc. (Eng.) (*Lond.*), Stud. I.C.E.
John Harold Gould Cook, Stud.I.C.E.
David Cooper, B.Sc. (Eng.) (*Lond.*).
Edward William Crack, Stud.I.C.E.
Frederick Harry Craven, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Alan Cronshaw, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Robert Norman Cross, B.A., B.A.I. (*Dublin*).
George Ronald Curtis, B.Sc. (*Edin.*).
Ronald Hawkins Dapling.
Reginald Brian Davenport, B.Sc. (*Manchester*), Stud.I.C.E.
Michael Henry Davis, Stud.I.C.E.
Lawrence Leonard Daw, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Alan Jackson Dean.
James innes Maclear Dempster, B.Sc. (*Bristol*).
Ronald John David Dixon, B.Sc. (*Manchester*), Stud.I.C.E.
Peter John Drury.
Alexander Dennis Eaton, B.A., B.A.I. (*Dublin*).
Peter Robert Evans.
Robert Oakes Evans, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
John Basil Fairhall, B.Sc. (Eng.) (*Lond.*).
William Ferguson, B.Sc. (Eng.) (*Lond.*).
Leonard Frederick Fish, B.Sc. (Eng.) (*Lond.*).
John Robert Fitzmaurice, B.E. (*New Zealand*).
Walter James Loudon Gallie, B.Sc. (*Glas.*).
Kenneth Michael Gammon, B.Sc. (*Birmingham*).
Ivor Peter Gillson, B.Sc. (Eng.) (*Lond.*)
John Reid Goldie.
Edward Gourley, B.Sc. (*Belfast*), Stud. I.C.E.
Jack Emile Grbic, B.E. (*New Zealand*).
William Arthur Green, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Colin Michael Guilford, B.A. (*Cantab.*), Stud.I.C.E.
John Edward Gwinnett.
William Hopton Hadley, Stud.I.C.E.
Ian Wilson Hannah, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Rosslyn Bernard Darrell Harris, B.Sc. (Eng.) (*Lond.*).
John Hartley, B.Sc. (Eng.) (*Lond.*).
William Brian Haslam, B.A. (*Cantab.*).

Alan Laurence Hider.
Owen Higby.
Ronald Royle Higgins.
Brian Edmund James Hook, B.Sc. (Eng.) (*Lond.*).
Ian Ross Hooper, B.A. (*Cantab.*), Stud. I.C.E.
Ronald Isaac Hopkins, B.Sc. (*Aberdeen*), Stud.I.C.E.
Thomas King Horne, Stud.I.C.E.
Philip Richard Stephen Houghton, B.Sc. (*Birmingham*).
Norman Howard, Stud.I.C.E.
Eric George Auguste Humphreys.
Roy Massey Hutson, B.A., B.A.I. (*Dublin*).
John Hampden Hyatt, B.A. (*Cantab.*), Stud.I.C.E.
Arthur Hyslop, Stud.I.C.E.
John Arthur James, B.Sc. (*Bristol*).
Dennis Edward Jared.
Isaac David Hubert Jenkins, B.Sc. (Eng.) (*Lond.*).
John Vincent Jeffreys Jones.
Vyvyan Philip Jones, B.Sc. (Eng.) (*Lond.*).
Philip Robert Henry Kelley, B.Sc. (*Durham*).
William Herbert Kershaw, M.A. (*Cantab.*).
Robert Ian Kinnear, B.Sc. (*Edin.*).
Gilbert Wilson Kneen.
John Keith Levitt, B.Sc. (*Leeds*), Stud. I.C.E.
David Charles Lewis, B.Sc. (Eng.) (*Lond.*).
Anthony George Lickfold, B.Sc. (Eng.) (*Lond.*).
John Barr Lightbody, B.Sc. (*Belfast*), Stud.I.C.E.
Michael Peter Littlejohn, B.Sc. (*Aberdeen*).
Pieter Botha Loedolff, B.Sc. (*Cape Town*).
Edward Ernest Lyons, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
John McBrien, Stud.I.C.E.
Harold Thompson McCaughan, B.Sc. (*Belfast*).
Ronald Pollock McCulloch, B.Sc. (*Glas.*).
Ian Hugh McIntyre, Stud.I.C.E
Maurice Edgar McKee, B.Sc. (*Belfast*), Stud.I.C.E.
Douglas Stoddart Mackenzie, B.Sc. (*Glas.*), Stud.I.C.E.
Patrick Anthony MacSweeney, B.E. *National*).
Penry Mainwaring, B.Sc. (*Wales*), Stud.I.C.E.
Richard Henry James Manley, Stud. I.C.E.
Edwin David Marden, B.A. (*Cantab.*).
Charles Williams Marshall.
Philip Anthony Martinez, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
John Edmonds Marwood.
Eric William Messer, Stud I.C.E.
Frank James Moulden.
Bernard Healy Newman, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Ernest Clive Newman, B.Sc. (Eng.) (*Lond.*).
Archibald Nisbet, B.Sc. (*Glas.*).
Robert Alexander Logie Nish, B.Sc. (*Edin.*), Stud.I.C.E.
John Kerry O'Mahony, B.E. (*National*).
Francis James Otten, B.A. (*Cantab.*).
Richard Godfrey Oxborrow, B.Sc. (Eng.) (*Lond.*).
Charles Alan Page, B.Eng. (*Liverpool*), Stud.I.C.E.
Derek Richard Parker, B.Sc. (Eng.) (*Lond.*).
Roy Albert Peel, B.Sc. (*Birmingham*), Stud.I.C.E.
George Kenneth Perks, Stud.I.C.E.
Anthony Percy Perry, B.A. (*Cantab.*).
Alan Ernest Peterson, B.Sc. (*Nottingham*).
Ian Booth Petrie, B.Sc. (*St Andrews*).
Arnold Boyd Phillips, B.A. (*Cantab.*).
Denys Reynolds Morley Pinfold, B.E. (*New Zealand*), Stud.I.C.E.
Eric Ross Posselwhite, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Allan Edward Pratt, B.Sc. (Eng.) (*Lond.*)
Jack Price.
David Barclay Ramsay, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Eric Rawcliffe, B.Sc. (*Manchester*),
Norman Arthur Read, B.Sc. (Eng.) (*Lond.*) Stud.I.C.E.
Walter Redlich, B.Sc. (Eng.) (*Lond.*).
Wallace John Reid, B.A.I. (*Dublin*).
Arthur Hugh Clarke Richards, B.C.E. (*Melbourne*), Stud.I.C.E.
Gerald William Richards, Stud.I.C.E.
Glyn James Riddell, Stud.I.C.E.
Munro Riddler, B.Sc. (*Aberdeen*).
Gerald Anthony David Roberts, B.Sc. (Eng.) (*Lond.*).
Alexander Stuart Robertson, B.Sc. (*Glas.*).
Geoffrey Arthur Robinson.
Geoffrey William Robinson, M.A. (*Oxon.*).
Brian Anthony Rogers, B.Eng. (*Liverpool*), Stud.I.C.E.
Gordon Victor Rollinson, B.Eng. (*Liverpool*), Stud.I.C.E.
Clifford Harrison Rothwell, Stud. I.C.E.

David John Russell.
David Minton Russell, Stud.I.C.E.
Roy Albert Sampford, Stud.I.C.E.
Kenneth Ashleigh Sankey, B.Sc. (Eng.) (*Lond.*)
Amarasinghe Aratchige Don Oliver Paul Saparamado, B.Sc. (Eng.) (*Lond.*).
Dennis Scothern.
Andrew Fraser Scott, B.Sc. (*Edin.*), Stud.I.C.E.
Joseph Rienzi Vistrine Seneviratne, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Bernard Gladstone Sharman, B.Sc. (Eng.) (*Lond.*).
Gordon Douglas Sharpe, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Michael Thomas Shilston, B.Sc. (Eng.) (*Lond.*).
Geoffrey Hardwick Simons, B.Sc. (*Manchester*), Stud.I.C.E.
Edward Skelton, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Peter Philip Smith, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Wilfred Collier Smith, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Terence Alexander Stoker, B.Eng. (*Liverpool*), Stud.I.C.E.
Reginald George Stone, Stud.I.C.E.
Basil Richard Streeten, B.A. (*Cantab.*).
Maurice Arthur Tester, B.Sc. (Eng.) (*Lond.*).
Norman Thomas, B.Sc.Tech. (*Manchester*).
John Cameron Thomson, B.Sc. (*Glas.*).
Derek William Tollerfield, B.Eng. (*Sheffield*).
Anthony Elliott Tumim.
Peter Valente, B.Sc.Tech. (*Manchester*).
John Ward, B.Sc. (*Nottingham*), Stud. I.C.E.
John Kinnear Waterhouse, B.Sc. (*Durham*).
Kenneth MacDonald Watt, B.Sc. (Eng.) (*Lond.*).
Brian Hunter Watters, B.Sc. (*Belfast*).
John Alexander Webster, B.Sc. (Eng.) (*Lond.*).
Keith Edward Marcus White, Stud. I.C.E.
John Haslop Whittaker, B.Sc. (Eng.) (*Lond.*).
Francis Christopher Wild, B.Sc. (Eng.) (*Lond.*).
Anthony Arthur Butler Williams, B.Sc. (*Witwatersrand*).
Andrew Wilson.
David Arthur Wright, M.Sc. (Eng.) (*Lond.*). Stud.I.C.E.
Lewis Bryant Yates, Stud.I.C.E.
Andrew Harvey Clare Young, Stud. I.C.E.
Robert Leslie Young, Stud.I.C.E.
Varusai Miskin Shamsuddin Ziauddin, M.Sc. (Eng.) (*Lond.*).

and had admitted as

Students

George Frederick Ackroyd.
Thomas Addison.
Robert Chappell Ainslie.
Frederick Grant Allan.
Douglas James Anderson.
John Bartley Archer.
George Baillie.
David Lee Benham, B.A. (*Oxon.*).
Peter Edwin Owen Bentley.
Graham Lloyd Benton.
Howard Simon Charles Bestow.
Alastair Ross Biggart.
William Stuart Bourne.
Michael Joseph Bramson.
Harold William Menos Brewer.
Archibald Wilson Cameron.
John William Edwin Campbell.
Eric Carruth.
Waldemar Ceglowski.
John Michael Chadwick.
Alan Ferguson Chalmers.
John Henley Chick.
Leonard Ernest Alfred Clark.
John Michael Clayton.
James Edward Cocksedge.
Peter John Collier.
Peter Abbey Johnson Collinson.
Derek Collis.
Alexander Coull.
Richard Robert Edward Dath.
Anthony Hubert Deane.
Joseph Peter D'Lord, B.Sc. (*Bombay*).
Peter Kenyon Donaldson.
Peter Michael Downing.
Gordon Ray Duff.
Peter Learmonth Du Heaume.
William Richard Faithful.
Keith Falconer.
Terence Hugh Farrell.
John Hurst Featherstone.
Yenon Israel Fellman.
David John Francis.

John Trevor Fulton.
Clifford Henry Gale.
Brian Robert Gibbins.
John David Wailes Green.
David Anthony Greenwood.
Bramwell Logan Gregson,
Derek Grimshaw.
Nahum Grunhut.
John Graeham Hardman.
John Robert Harvey.
Barry Albert Haseltine.
John Murray Hawkes.
Anthony Oliver Hazzard.
Russel James Hodge.
Roger Holden.
Colin George Holmes.
John Aitken Horne.
William Braidwood Jack.
William John Jardin.
James Wright Jenkins.
Roger Paul Johnson.
Chandler Jones.
Michael Champion Jones.
Michael Richard Kearney.
James Burton Kirch.
Samuel Francis Kwaku.
Maung Maung Kyaing.
James Sloan Lamont.
John Gordon Leeney.
David John Lewis.
John Marteine Lewis.
Michael John Lloyd.
Anthony Owen Lucas.
David McDiarmid,
Frederick Harold McHenry.
Alister Iain Maclean Macmillan.
Patrick Hugh McMillan.
Garry MacNaughton.
Chinniah Mahendran.
Menasé Massias.
Vinod Bansi Mathur.
William Albert Metherell.
Roger Glyn Morris.
Peter Moses.
Michael Edward Mountain.
James Park.
Ralph Andrew Bruce Paterson.
Thomas Patterson
Paul Tremain Pearce.
David Gwyn Powell.
Leslie Anthony Probert.
Kenneth Andrew Pye.
James Purves Fraser Redden.
Brian Vincent Redknap.
Robert Reid.
Gordon Campbell Rendall.
Tony Melville Ridley.
Antony Richmond Ritchie.
Eric Roberts.
Michael Charles Roberts.
Brian Morley Rodmell.
Trevor Seaman.
Richard Charles Setchfield.
Ian James Short.
Raymond Stead.
John Northedge Stothard.
Frank Taylor.
Stanley Taylor.
James Easten Tyson.
John Joseph Wadley.
Leslie James Walker.
Malcolm John Wardman.
Desmond John Watson.
John Hubert Innes Watt.
Arthur Kenzie Webb.
Gerald Richard Webb.
Brian Wilkinson.
David Gordon Wilson.
John Frederick Winfield.
Brian Gordon Woodward.

The following Paper was presented for discussion, and, on the motion of the President, the thanks of the Institution were accorded to the Authors.

Paper No. 5912

"The Claerwen Dam"

by

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SYNOPSIS

The Paper describes the design and construction of a mass concrete gravity dam about 200 feet high and 1,066 feet long. The dam was built with mass concrete hearting, masonry facing on the downstream and upper portion of the upstream faces, and blue-brick facing on the remainder of the upstream face.

The foundation of argillaceous shale was not ideal and pressure grouting was necessary to seal the interstices to a depth, in general, of 50 feet.

A 7-to-1 mix was specified for the concrete hearting and 6-inch aggregate and low-heat cement were used. The use of these gave rise to new problems, particularly in regard to harsh aggregates and setting in cold weather.

Scale models were constructed and operated on site to decide the best form of spillway crest, side spillweir channels, and measuring weir, and these are described.

Shortage of steel and timber necessitated the re-design of the reinforced-concrete roadway arches over the dam and the substitution of precast, prestressed arches, and this somewhat unusual work is described.

In Part 3 of the Paper, the plant and constructional methods used by the contractor are described in some detail.

Historical

In 1892, on the recommendation of Mr James Mansergh, Past-President I.C.E., the City of Birmingham obtained Parliamentary powers to develop the catchment area of 45,562 acres on the Rivers Elan and Claerwen in central Wales (see *Fig. 1*) by the construction of six reservoirs to impound 17,250 million gallons. As a first instalment, three reservoirs on the River Elan, together with an aqueduct, filtration plant, and service reservoirs were completed and brought into use in 1904.[1]

These works were estimated to give a reliable yield of about 72 million gallons per day, of which 45 million gallons per day would be made available

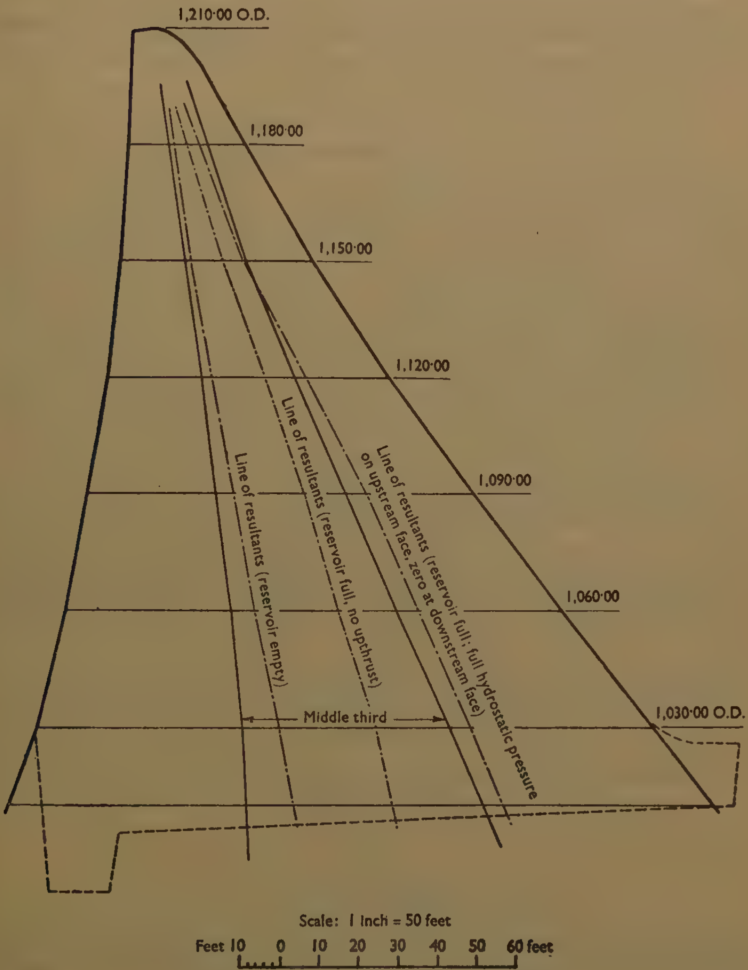
[1] E. L. and W. L. Mansergh, "The Works for the Supply of Water to the City of Birmingham from Mid-Wales." Min. Proc. Instn Civ. Engrs, vol. 190 (1911–12, Pt IV), p. 3.

Fig. 1

Location Plan

to the City, and 27 million gallons per day for compensation water. By 1937, the average daily consumption had risen to 34 million gallons and was increasing by 1 million gallons per day each year so that it was evident that the contemplated further development of the catchment was due.

Fig. 3



THEORETICAL DESIGN SECTION

The original plan was reviewed in the light of modern practice and it was found that the full economical yield of the Claerwen Valley could be

obtained by the construction of a single dam, 200 feet high, at Cerig Cwplau (see Fig. 18, Plate 2) in place of the three smaller dams originally planned.

Powers were obtained, therefore, under the Birmingham Corporation Act 1940 to vary the original scheme by the construction of one large dam to impound 10,625 million gallons, thus increasing the total storage to 21,800 million gallons and the reliable yield available to the City to not less than 75 million gallons per day. Owing to the war, work could not be started until 1946.

Sir William Halcrow and Partners were appointed Engineers for the scheme, and the Contract was let to Edmund Nuttall, Sons & Co. (London) Ltd.

General Description

The long-period average annual rainfall over the catchment is 69·4 inches (*Fig. 14*), the catchment area above the Claerwen Dam being 13,260 acres. With top water level at 1,210 feet above O.D., the reservoir covers 650 acres with a maximum depth of water of 184 feet and the volume impounded is 10,625 million gallons. The quality of the moorland water is distinctly acid with a seasonal variation from 4·5 to 6·0 in *pH* value.

Geologically the catchment area consists of slates, grits, and conglomerates of the Lower Silurian order, and the new dam, as in the case of the two upper dams on the River Elan, is founded on an argillaceous shale.

Evidence from trial pits and boreholes, later confirmed by the excavation, proved the formation to consist of a slate or very dense shale weighing 175 lb. per cubic foot, dipping from north-north-east to south-south-west at about 15 degrees, but parting easily on cleavage planes normal to the natural beds (see *Fig. 2*, facing p. 270). The rock outcrops or is lightly covered with soil, clay, or peat over most of the foundation area except on the west bank, where a depth of up to 20 feet of soft material overlay the rock.

A fault from 4 to 8 feet wide, hading about 19 degrees, follows generally the line of dip and intersects the foundation of the dam on the west side.

Part 1.—Design

by

H. D. Morgan, M.Sc.(Eng.), M.I.C.E.

The original dams were built of cyclopean rubble, faced with rock-faced block-in-course gritstone masonry. A number of profiles for the new dam were prepared and considered, and that selected is shown in *Fig. 3*. This design ensured that the vertical component of the pressure on the foundations nowhere exceeds 10 tons per square foot—a design condition adopted in view of the nature of the foundation rock.

The profile of the cross-section of the dam resulting from the adoption of a constant maximum vertical stress of 10 tons per square foot was determined initially from a formula obtained by analytical methods. This expression which relates the base width to the depth below maximum water level at any horizontal plane was derived in the manner shown in the Appendix by a method suggested by Professor A. H. Naylor. The analysis assumes trapezoidal distribution of stress on the horizontal plane. The profile was checked in the usual way by calculation and graphical methods. In the upper levels the profile does not conform with the formula, the widths being greater than those calculated in order to meet practical requirements.

The assumption of trapezoidal stress distribution is, of course, not strictly correct, although quite sufficiently so for preliminary design. However, a more exact stress analysis was carried out by a method developed by Dr O. C. Zienkiewicz—at that time a member of the Authors' firm. This treatment is very interesting, being analogous to the relaxation method for analysing indeterminate structures.

Designed as a gravity dam, curved in plan to 2,000 feet radius for appearance (see *Figs 4* and *5*), no credit for any arching effect was given in the calculations. The maximum head of water assumed was 3 feet above flood crest level, the weight of concrete being taken as 140 lb. per cubic foot. Full hydrostatic pressure at the upstream face and zero at the toe was assumed for uplift.

This assumption was conservative since it took no account of the effect of the relief pipes. These pass from a point immediately downstream of the cut-off up to the inspection galleries. The pipes are 4 inches in diameter and placed at about 15-foot centres.

The spillway, situated centrally and designed for a maximum flood of 5,800 cusecs at 3 feet depth over sill, is 540 feet long and consists of six lengths of 40 feet on either side of a central 60-foot length, the level of which is 6 inches below the main spillway and is designed to take normal low flows up to about 70 cusecs.

For architectural effect, and also to retard the flow into the stilling-pool and gauge-basin, the downstream face below the central crest and the floors of the side spillweir channels are stepped, whilst the remainder of the downstream face is plain. (See *Figs 4* and *5*.)

Foundations

The foundation of 180 feet maximum width is benched into the hillsides, and at the downstream side is carried 10 to 15 feet into sound rock and sloped down at 1 in 24 towards the upstream side (*Fig. 6*). The cut-off trench varies in width and depth from 15 feet at the centre to 6 feet at the ends of the dam, and its depth below the top of sound rock ranges from 30 feet on the east side to more than 60 feet on the west, where the ground

contours lie somewhat unfavourably. A maximum depth of excavation of 75 feet below ground level occurs on the west-bank side.

Two samples of rock from the foundation were dressed into 6-inch cubes by a mason and then were tested in compression in a laboratory. The first sample tested across the cleavage planes withstood the maximum capacity of the testing machine, 6,350 lb. per square inch (404 tons per square foot), but the second sample, tested in line with the cleavage planes, failed at 3,545 lb. per square inch (228 tons per square foot).

Considerable difficulty was experienced in maintaining the desired shape of the main foundation excavation and the cut-off trench in the smaller sections near the ends of the dam, owing to the way in which the rock was shattered by blasting and peeled away on the cleavage planes (*Fig. 2*). As a result, overbreak was, in places, considerable. In the larger sections at the centre of the valley there was much less difficulty.

The area of foundation on the west bank which was faulted was closely investigated. The fault was of considerable age and depth and the rock was loose near the surface at each side of the fault, which was about 7 to 8 feet wide at this level and clay-filled. The condition of both the clay and the adjoining rock became harder and denser as the depth increased, and the width varied and decreased to 4 or 5 feet. Well above foundation level the rock sides were sound and the material in the fault became a near-shale which could be loosened only with pneumatic tools.

Test holes, drilled diagonally, proved that the conditions were similar down to at least 80 feet below rock surface, and it was found that no grout was accepted by them. For a distance of about 700 feet upstream of the dam the river bed coincides with the line of the fault, and in spite of this, no water at all seeped into the excavation from the fault material itself or the rock adjoining, down to 60 feet below rock surface, to which level the cut-off trench was taken at this point. Additional grout-pipes were inserted into the rock on each side of the fault in the trench bottom and also into any apparent fissures in the upstream face near the top of the excavation. Apart from this, no further precautions were deemed necessary, since the clay in the fault was quite impervious and no leakage has been observed since the reservoir was filled.

Design Details

The dam is divided into twenty-three radial blocks, each 46 feet 6 inches wide (Figs 7, Plate 1) and, during construction, these were raised alternately, the concrete in the leading blocks being kept 4 weeks in advance of that in adjoining closer blocks (*Fig. 10*).

Two inspection galleries are provided in the dam, one horizontal at 35 feet below crest and the other stepped to follow generally the ground level of the valley. They connect, from road level at each end, to two valve chambers in the heart of the dam, access to which is also provided by

12-foot-diameter tunnels from the two valve houses at the toe of the dam. A 6-inch open-jointed drain in shallow trench in the foundation just downstream of the cut-off trench is connected to the galleries by 4-inch-diameter pressure-relief holes formed in the concrete. Similar holes connect lower and upper galleries at about 15 feet spacing, and are extended above the latter to 10 feet below the crest. Seepage is collected in sumps in each valve chamber and discharged into the downstream end of the centre spillweir channel through 12-inch-diameter drains.

Galleries, valve houses, and chambers are lit by electricity generated by two 1·8-kilowatt 240-volt D.C. generators driven by 3-horse-power Pelton wheels. The Pelton wheels are operated from the head in the reservoir, through pipes taking off from the by-pass to each upstream sluice valve on the draw-off mains.

Draw-off is provided by two 48-inch-diameter pipes laid in the tunnels between the valve chambers and the valve houses at the foot of the dam (*Fig. 4*). The pipes discharge into the stilling-pool through needle valves with drowned bellmouth outlets. Each pipe divides near the upstream end into two pipes, each of 36 inches diameter, which pass through the upstream face of the dam to give draw-off at two levels (*Figs 8* (*a*) and *10*). Two 24-inch-diameter sluice valves, one wedge faced and one paralled-faced, are placed in each pipe in the valve chambers. Bronze self-clearing screens protect the intakes at the upstream ends.

Scour is provided for at a lower level by two 36-inch-diameter pipes also controlled by wedge-faced and parallel-faced sluice valves in the valve chambers. These scour pipes discharge into the downstream end of the centre spillweir channel.

All valves are manually operated through gearing. Mechanical operation, by hydraulic ram or electric motor, was considered and rejected, partly on account of cost and partly on account of space limitations, but principally because of the difficulty of maintenance in such an isolated site and because manual operation easily provides the maximum speeds of opening and closing required. The pipes are of steel plate, lined with spun concrete, with bitumen painting internally. Joints are either flanged or of spigot-and-socket type with run lead.

Facing

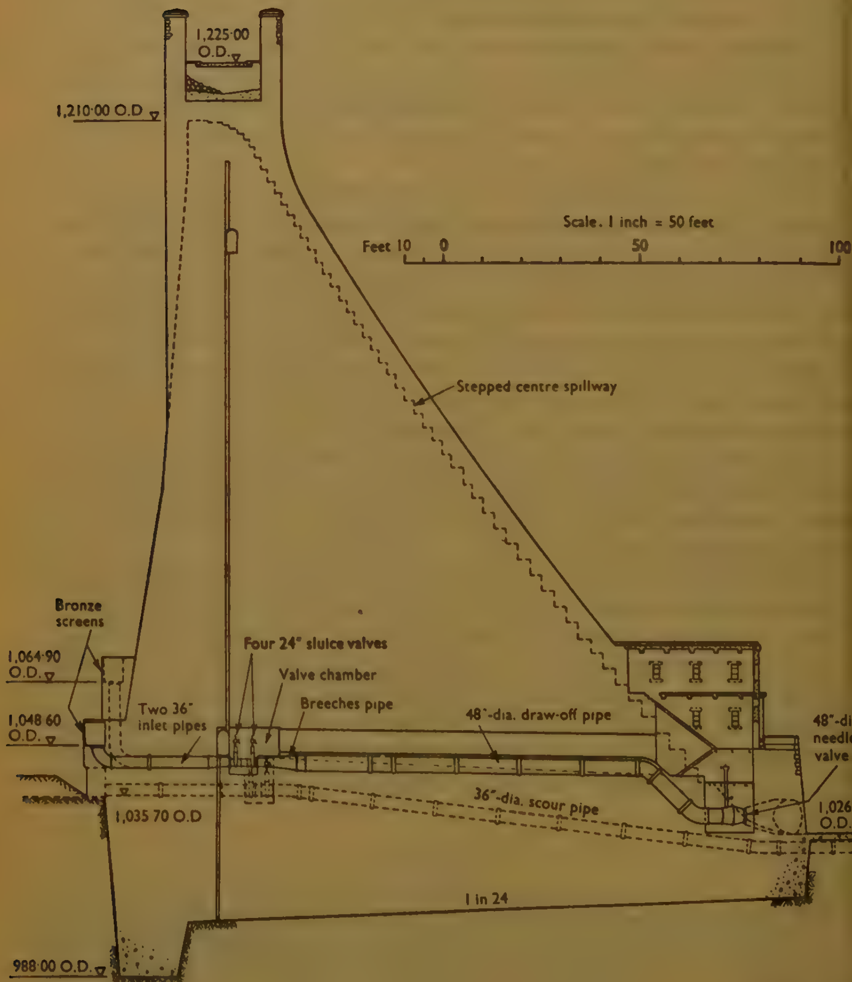
The Birmingham Corporation were anxious that the new work should harmonize and be in keeping with the high standard of the old dams. Accordingly, when tenders were invited from selected firms in 1946, the contractors were asked to quote for three alternative forms of construction as follows :—

(*a*) A concrete dam with brickwork and rock-faced masonry facing set in precast blockwork.

(*b*) A concrete dam with brickwork and masonry facework as above but all built in situ.

(*c*) A concrete dam with brickwork and ashlar masonry as above, the downstream facing to be in fine concrete.

Fig. 8 (a)



CROSS-SECTION OF DAM ALONG 48-INCH DRAW-OFF PIPE

In each case the road parapets and valve houses were to be in fair-tooled ashlar (*Fig. 4*). The second alternative was adopted.

The upstream face of the dam from the surface of the rock to a point 30 feet below top water level is faced with Staffordshire brindle engineering brick (*Fig. 10*), the average thickness of brickwork being 13½ inches. The brickwork was erected in stages of 4 feet at a time to suit the concrete lifts,

Fig. 8 (b)

Cross-Section of Dam through Block 9

for which it formed the upstream shutter. Since this face, on the bottom section of the dam, is on a batter of 1 in 6, some difficulty was experienced

in wet weather in overcoming a tendency for the un-frogged brickwork to slide on its bed, especially if progress was too rapid.

Above the brickwork, the rest of this face, the whole of the downstream face, and the channel retaining walls and bridges were faced with squared rock-faced random rubble gritstone masonry of an average thickness of 9 inches, about one-sixth of the face area consisting of header stones 18 inches on the bed.

The Contractor proposed to use only a Derbyshire gritstone for the facing, but it soon became evident that the rate of supply from the quarries supplying this would be quite inadequate, and in 1949 it was necessary to extend the source of supply to the Blue Pennant quarries in South Wales. To ensure an adequate rate of delivery, even with this additional source, it was necessary to import masons from Northern Italy and more than a hundred Italians were employed, about twenty-seven of them as fixers on the site and the remainder for dressing the stone in the quarries.

Whilst the efforts of the Contractor and suppliers in tackling the difficult problems of recruiting and maintaining the Italian labour force are fully acknowledged, it must be recorded that the more formidable task of overcoming Government and Trades Union objections to the use of foreign labour could not have been achieved without the determination and negotiating ability of Mr A. E. Fordham, General Manager and Secretary of the City of Birmingham Water Department, who carried out the wishes of his Committee so successfully that permission was obtained and the work completed without any trouble or the loss of goodwill on either side. Tribute must also be paid to the Trade Union concerned for accepting the position and withholding their objections.

Since the two stones are of quite different colour it was necessary to decide whether to mix them in the work, or to build different sections of the work entirely in one type of stone. The latter course was chosen and the Derbyshire stone was used to face the spillweir channel retaining walls, the footbridge crossing the stilling weir, and the upstream face of the dam below top water level. The whole of the rest of the work was faced in Blue Pennant stone.

In view of the delay involved in obtaining the masonry, the Contractor was permitted to proceed with the concreting of the dam with a temporary shuttered face (*Figs 6*, *8* (*b*), and *19*) on the downstream side, the masonry (together with a small quantity of concrete backing) being added when it became available.

Parapets and door and window surrounds are of fair-tooled ashlar masonry (*Fig. 4*).

Concrete backing the upstream brickwork and masonry and embedding the contraction-joint copper strips is of 4-to-1 quality with 1½-inch-maximum-size aggregate; the thickness of the backing ranges from 6 feet at the bottom to 3 feet at the top of the dam. Cut-off-trench concrete is of 5-to-1 quality with 3-inch-maximum aggregate, and the whole of the

hearting concrete is of 7-to-1 quality with 6-inch-maximum aggregate. All the above concrete was made with low-heat Portland cement manufactured in South Wales.

Road Viaduct and Bridges

The road viaduct and all bridges were first designed in normal reinforced concrete with masonry voussoir stones attached on each face, but owing to difficulties in obtaining reinforcing steel and timber for centering a design in prestressed concrete was adopted for the thirteen arches in the viaduct over the spill-weir crest.

A 75-foot-span road bridge over the river downstream of the site and also three footbridges (*Figs 4* and *5*) over the spillweir channels were constructed as designed originally in reinforced concrete.

A stilling weir and a gauging weir are placed across the centre spill-weir channel and are built in plain concrete.

Part 2.—Construction

by

P. A. Scott and R. J. C. Walton, MM.I.C.E.

River Diversion

During the early construction (*Fig. 9*), the river was diverted by temporary dams into a channel about 820 feet long and of 100 square feet cross-sectional area, excavated on the east side of the original river bed. This allowed for the construction, to above flood level, of the three central blocks of the dam which contained the scour mains, valve chambers, and 12-foot-diameter tunnels. Very little water was encountered in the excavation, and only on rare occasions was the capacity of the diversion channel exceeded by floods.

The diversion channel was then closed at the upstream end by a third temporary dam (*Fig. 10*) at a period of very low river, and the flow then passed through the two scour mains and, in times of high flow, the east tunnel which was left open for this purpose.

On completion of the excavation and raising of the concrete over the area occupied by the diversion channel, the upstream end of the east tunnel was plugged with concrete and grouted.

Pressure Grouting

Drilling and grouting below the cut-off trench, to seal the foundations, was carried out after concreting the trench, steel pipes being set in the concrete for the purpose (*Fig. 2*).

Holes of 1¾ inch diameter at 6-foot centres were drilled in two lines about 6 feet apart, water-tested, and grouted in stages. Diagonal and horizontal holes were also drilled at the wings of the dam and in a few other places to test the results of the vertical grouting.

Holes were generally 50 feet deep below cut-off bottom and were drilled and grouted in four stages, additional stages being drilled if necessary, and a few holes were taken initially as deep as 100 feet to test the ground.

Test and grouting pressures were equivalent to 1½ times the static head on the bottom of the hole with full reservoir. Bottoms of the pipes were set a few inches above rock to ensure the grouting of the junction of the cut-off concrete and rock. About one hole in four was treated as a primary hole and grouted before the remainder.

Fissures generally were very fine, and grout consistencies as thin as 20-to-1 water/cement were used frequently. Experiments were made to study the effect of pre-injecting silicate of soda but they showed that no benefit could be obtained.

Whilst results naturally varied from area to area, on an average 60 per cent of the grout was accepted in the first two stages, 35 per cent in the third and fourth stages, and 5 per cent in the deeper holes.

Two-inch grout pipes were large enough to pass the boring tackle, but trouble arose from pipes becoming bent by contact with skips during concreting and it was found better to use 3-inch-diameter pipes, which were stouter and were still usable after slight distortion.

When a pipe had to be abandoned a new hole was drilled in the concrete alongside and a short pipe grouted into it; normal procedure was then resumed. On two occasions there was some evidence of the grout tending to lift the upper layer of concrete when using the above method. In these cases grouting was immediately stopped and the hole was drilled right down to the full depth of 50 feet in one operation, the steel pipe being extended upwards and grouting carried out only after additional concrete had been placed.

The total drilling involved below cut-off bottom was 18,040 linear feet, of which less than 800 feet was at depths below 50 feet. A total of 66 tons of cement was used in these holes, of which 15 tons was required to fill up holes after completion of the pressure grouting. In addition to the above, a total of 15 tons of cement was injected into a few surface fissures in the broad foundation and adjoining the fault.

Concrete Aggregates

The contractor elected to open and work a quarry (*Figs 1* and *20*) on Corporation property for the bulk of the aggregate required for concrete. The area selected consisted of a quartzitic gritstone from the Lower Llandovery beds, weighing 168 lb. per cubic foot, which was situated

about 4 miles from the dam site. About 410,000 tons of stone was produced from this quarry and about 60,000 tons of middle and small sizes of aggregate was obtained from local quarries at Rhayader and Builth Wells.

A maximum size for aggregate of $2\frac{1}{2}$ inches had been specified for the mass concrete, but at the Contractor's request this was increased to 6 inches, and the final arrangement provided for four grades: 6 inches to 3 inches; 3 inches to $1\frac{1}{2}$ inch; $1\frac{1}{2}$ inch to $\frac{3}{16}$ inch; and sand below $\frac{3}{16}$ inch. (See *Figs 11.*)

In the Authors' opinion the further sub-division of the $1\frac{1}{2}$-inch-to-$\frac{3}{16}$-inch grade would have been beneficial by allowing closer control of the troublesome "pea" sizes, but this was impossible without major alterations to the weigh-batcher.

Low-Heat Portland Cement

All concrete within the profile of the dam up to the crest level and, beyond the limits of the spillweir, to a level 5 feet above crest, was made with low-heat cement.

A true low-heat cement is a very finely ground Portland cement of specially adjusted compound composition, the grinding approximating to that of rapid-hardening cement. It varies from ordinary Portland and rapid-hardening cements in that the proportions of the major components of the cement—tricalcium and dicalcium silicates—are adjusted by a restriction of the former and an increase in the latter, and the proportion of tricalcium aluminate may also be reduced by the partial substitution of iron oxide.

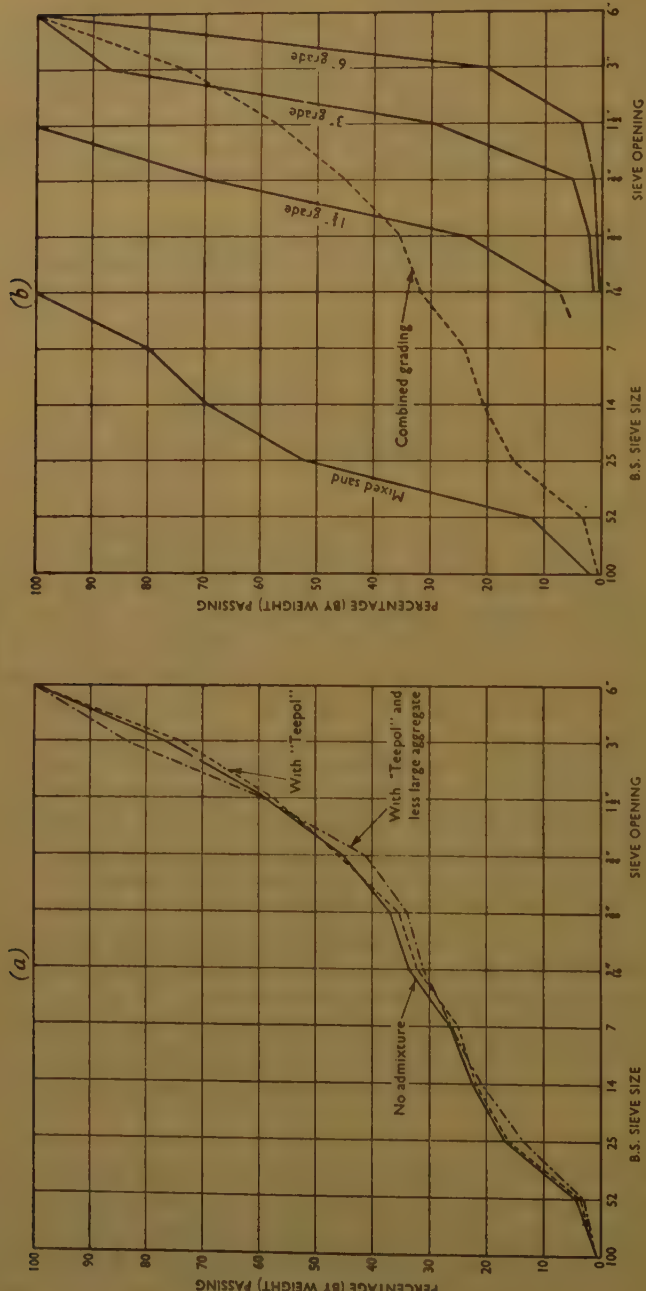
At the time the contract was let, British Standard No. 1370 of 1947 had not been issued, but all the cement was manufactured to this specification. One bin of 3,000 tons capacity was set aside at the manufacturer's works for the sole use of the Claerwen contract, and the cement was delivered to the site in bulk containers holding 3 to $3\frac{1}{2}$ tons.

For testing, sampling of the cement was carried out at the works by a representative of the testing laboratory, samples being taken over the several days duration of a grinding. These were then mixed and quartered to obtain a test sample to represent that grinding.

At the start of the contract the first samples covered grindings of 250 tons, but these were soon increased to about 1,000 tons each. In all, sixty-nine samples were tested representing 61,835 tons of low-heat cement supplied.

Considerable variations were evident in the vibrated-mortar cube-strengths (Fig. 12, Plate 1) and, whilst all samples tested satisfied the specification for fineness, heat of hydration, and compressive strengths at the ages of 3 and 7 days, eight of the sixty-nine samples failed to reach the specified 28-day strength. It was suggested that that was due to the behaviour of the dicalcium silicate component of the cement, whose influence

Figs 11



Aggregate Grading Curves

on the strength starts about 7 days or more after hydration and has become fairly fully operative after 90 days.

At 28 days, however, it may still vary considerably. The strength of the resulting concrete, especially the lean mixes, consequently varied to a similar degree.

TABLE 1.—VIBRATED-MORTAR CUBE RESULTS FOR LOW-HEAT PORTLAND CEMENT

Age: days	Specified minimum strength: lb./sq. in.	Actual strengths	
		Minimum: lb./sq. in.	Maximum: lb./sq. in.
3	1,000	1,170	2,510
7	1,600	1,600	3,320
28	3,750	3,350	6,120

In view of the time taken to complete the full range of tests laid down in the new specification, the cement had always been used in the work before the test results were available on site. Additional samples were therefore taken from the silos on site and tested in Birmingham for strength and setting-time only, 7-day results being telephoned to site to give early indication of quality. This method also gave warnings of damage in transit.

Difficulty arose during a prolonged wet period in the winter months of 1949 to 1950, from the cement becoming damp in transit, and, in a few containers, as much as 5 per cent of the cement was found to have set hard on arrival at site. Both the top and bottom hatches of the containers became distorted with use and were not proof against severe weather, and sheeting of rail wagons was difficult to ensure at all times owing to shortage of railway staff. It was equally important to ensure that cement was not filled into containers which had become wet inside while being returned empty. The above was the only cause for the rejection of certain quantities of cement during the contract.

LOW-HEAT-CEMENT CONCRETE

The three classes of concrete used in the body of the dam are shown in Table 2.

The strength specified in the contract applied only to normal Portland-cement concrete and at the beginning very little information was available as to the strengths to be expected from low-heat-cement concrete at early ages. It was decided to accept, for the low-heat-cement concrete, strengths at the age of 3 months equal to those specified for Portland-cement concrete of similar class at 28 days.

TABLE 2.—CONCRETE MIXES AND STRENGTHS

Class	Postion in dam	Proportion by volume : cement to mixed aggregate	Ring size passed by aggregate: inches	Approx. cement content per cubic yard of concrete : lb.	28-day strength for P.C. concrete : lb./sq. in.
B	Backing to up-stream face	1 : 4	1½	580	3,100
C	Cut-off trench	1 : 5	3	480	2,750
D	Hearting	1 : 7	6	360	2,300

On this basis, no difficulty was experienced in obtaining workable mixes of satisfactory strength with concrete of Classes B and C, but considerable difficulty arose at first with the Class D 6-inch mass concrete. A number of factors appear to have been responsible for the trouble.

Whilst there is some evidence that the compaction factor of concrete made with low-heat cement is less favourable than that of a concrete made with normal Portland cement, the major cause of the difficulty undoubtedly arose from the intractable nature of the aggregate and the crusher-run sand, which made it necessary to maintain a water/cement ratio in excess of 0·7 to obtain workability with the lean mix.

This encouraged the segregation of the large aggregate, which tended to occur at each handling operation ; that is, at discharge from mixer to the wet-hopper, from wet-hopper to skip, and at discharge of the skip.

Concrete was mixed in 3-cubic-yard batches and transported in skips holding 4 to 4½ cubic yards. Thus each skip contained more than one complete batch, and it is possible that one skip received an undue proportion of the coarse aggregate from the split batch while the following skip received a surplus of the remaining fines. Additional men were therefore required at the point of placing to distribute pockets of aggregate into the mass.

Some improvement resulted from the introduction into the mix of Severn river sand, which was used in a proportion, by volume, of not less than two-thirds of river sand to one-third of crusher-run sand.

Experiments were also made with the addition of the wetting agents "Lissapol N" (a non-ionic compound) and "Teepol 410" (an alkyl sulphate), trials being continued over several weeks.

Since the addition of the latter greatly improved workability whilst allowing a reduction in the water/cement ratio of up to 10 per cent, and in the sand content of up to 2 per cent, which practically eliminated the segregation of the mixed concrete, its use was adopted. The manufacturers later substituted the improved compound Teepol 530.

The Teepol was added to the mixing water at the rate of ½ pint per

3-cubic-yard batch (1·1 ounce per cwt of cement), and was used only in the Class D mass concrete containing low-heat cement.

Average results obtained from crushing standard 6-inch test cubes made from low-heat-cement concrete are listed in Table 3. Samples taken from concrete containing 6-inch stone were wet-screened through a 2-inch mesh before the cubes were cast.

TABLE 3

Class	Weight: lb. per cu. ft	Water/cement ratio	Compressive strength: lb. per sq. inch		
			7 days	28 days	90 days
B	151·4	0·53	1,420	3,090	5,550
C	152·0	0·59	1,190	2,450	4,800
D no admixture	150·7	0·71	780	1,680	3,660
D Teepol 410	147·8	0·64	660	1,450	3,250
D Teepol 530	148·7	0·64	870	—	3,490

Fig. 12, Plate 1, clearly indicates graphically how the Class D low-heat-cement concrete strength was influenced by variations in the strength of the cement itself.

DESIGN OF MIX FOR 6-INCH-AGGREGATE MASS CONCRETE

Some difficulty was experienced in arriving at a satisfactory mix for the Class D 6-inch mass concrete to give the maximum density, workability, and impermeability, and information on the use of a large-aggregate concrete in Great Britain could not be found. Information published in America and on the Continent was examined and theoretical grading curves suggested by Fuller and Bolomey were tried out.

Fuller's formula for maximum density grading of aggregate is:

$$P = 100\sqrt{\frac{d}{D}}$$

where P denotes the percentage by weight of aggregate finer than diameter d, and D the maximum diameter of aggregate.

Bolomey's formula for maximum density grading of aggregate is:

$$P = B + (100 - B)\sqrt{\frac{d}{D}}$$

where P denotes the percentage by weight of aggregate plus cement finer than diameter d.

d ,, the diameter of particle corresponding to P.

D ,, the maximum diameter of aggregate.

B is a constant. (Recommended values of B for a crushed-stone aggregate are: "plastic" concrete 12; "liquid" concrete 14.)

Both tests with these formulae, which give sand content of from 15 to 18 per cent, proved to be unworkable with the harsh aggregate available.

Experimental curves were then built up from some examples given for American works where 25 per cent of sand was used, but these also proved to be unworkable, and the minimum sand content was found by trial and error to be 33 to 34 per cent. The maximum quantity of the 6-inch-to-3-inch-grade stone which could be incorporated in the mix was similarly found to be about 25 per cent, but variations in the stockpiles made it desirable to limit this figure to about 20 per cent. An excess of large-size stone increased the tendency for the mix to segregate, but this, as previously mentioned, was later overcome to a large extent by the use of Teepol, which also allowed a reduction of the sand content to 31 or 32 per cent.

Further minor variations in the mix proportions were made from time to time to compensate for variations in the particle shape and the size of the aggregate, which arose through the use of stone from different parts of the quarry, and from changes in the crushing plant. Typical grading curves, for a mix with and without addition of Teepol, are shown in *Figs 11.* The fineness modulus of these gradings is about 6·8, but it was not found possible to use the fineness modulus as a means of control, since there were too many variables.

Having determined, by screen analyses, the actual proportions of each nominal grade required to give the desired composite clean grading of the aggregate, the quantity of cement per cubic yard of mixed concrete had to be ascertained to comply with the specified 1 : 7 proportion by volume of cement to mixed aggregate. The density of the dry mixed aggregate was found by hand-packing weighed amounts of each grade, in thin layers, into an 8-cubic-foot gauge-box, using the smaller grades to fill voids in the larger.

Thus, having found the density of the mixed aggregate as, say, 130 lb. per cubic foot, and with a cement density of 90 lb. per cubic foot, the mix would consist of 910 lb. of aggregate to 90 lb. of cement, the 910 lb. being sub-divided to give the desired composite grading as follows:

Cement	90 lb.	1 part
Sand below $\frac{3}{16}''$	273 lb.	3·03 parts
$1\frac{1}{2}'' - \frac{3}{16}''$ grade	200 lb.	2·22 parts
$3'' - 1\frac{1}{2}''$ grade	182 lb.	2·02 parts
$6'' - 3''$ grade	255 lb.	2·83 parts

Having found by experiment the specific gravities of the stone and sand, and taking that of cement as 3·16, Table 4 can be drawn up giving the absolute volume of all the ingredients, including the water, for a given water/cement ratio based on 100 lb. of cement.

TABLE 4

	Cement	Sand	Aggregates			Water
			$1\frac{1}{2}''$	$3''$	$6''$	
Parts by weight . . .	1	3·03	2·22	2·02	2·83	0·62
Weight for batch containing 100 lb. of cement .	100	303	222	202	282	62·0
Weight of 1 cu. ft solid .	197	164	168	168	168	62·4
Space occupied: cu. feet .	0·508	1·848	1·321	1·202	1·685	0·994
Total: cu. feet . . .			7·558			

Thus the weight of cement for 1 cubic yard is $\frac{27}{7{\cdot}558} \times 100 = 357$ lb.

Since the first line of Table 4 gives a direct proportion of the aggregate to the cement, the complete mix for any size of batch is quickly obtained.

It may be noted that, in dealing with a vibrated concrete with large aggregate, it was found essential to mix full-size batches to test for workability. Complete 3-cubic-yard batches were poured into 4-foot cube moulds made up from pressed-steel tank plates. When the mix was reasonably satisfactory the resulting blocks were later incorporated as plums in the work.

CONCRETE BATCHING AND PLACING

All concrete was proportioned by weight and mixed usually in 3-cubic-yard batches. The hearting of the dam was placed in lifts limited to 4 feet in depth and individual layers were limited to a maximum of 2 feet depth after consolidation by vibration. In summer, shutters were struck in the minimum time of 12 hours after completion of concreting, and generally not more than three 4-foot lifts were placed per fortnight on one area.

The programme allowed for placing an average of 3,000 cubic yards per week, and over a period of one year this average was exceeded, but screening difficulties at the start of the contract reduced the average over 3 years of concreting to 2,200 cubic yards per week. The maximum quantity placed in any period of 24 hours was 1,273 cubic yards.

No mechanical device was provided at the mixer for control of mixing time and consequently variations occurred. Check timing carried out at intervals showed that a period of 24 seconds was required to load and discharge the drum; that, on average, the ingredients remained in the

drum for about 83½ seconds; and that a batch was sometimes discharged from the drum after a mixing time of less than the specified 75 seconds.

Whilst the use of large aggregate did not appear to require a longer mixing time than would be necessary with a smaller aggregate, a large batch did seem to require more time than did a small batch and this was especially noticeable with the lean mix.

In this respect the Authors are of the opinion that a form of specification which relates mixing time to size of batch has much to commend it.

In order to reduce delay to concreting from the light frosts which were frequent, particularly at nights between December and May, hot water was used for mixing the concrete, and heating of the aggregate stocks was provided. No attempt was made, however, to carry out concreting during the infrequent short periods of severe frost.

It was found that with air temperature just below freezing point, if the mixing water was added at about 120° F., the temperature of the mixed concrete could be maintained above 40° F. until placed. After placing, covers were put on as soon as possible and the large mass of concrete itself then maintained a safe temperature.

Concrete Control and Testing

Samples of each grade of aggregate were taken daily from the horizontal conveyor drawing the materials from below the stockpiles, the belt being stopped while feeding a particular grade, and everything on a length of about 6 feet being removed into sample boxes. Final samples of 50 to 60 lb. weight of each grade were subjected to sieve analyses in the site testing room and, from the results of these, composite grading curves were plotted daily. Any marked variation arising from, say, the changing of jaws or wear of screens was thus immediately noticed, so that proportions of the various grades could be adjusted as necessary at the batcher.

Moisture in the sand was measured frequently by means of the Gammon-Morgan water-in-sand estimator and weights were adjusted accordingly. An allowance for moisture in the small and middle grades of aggregate was also made by visual means, based on extensive drying tests previously made on samples of varying degrees of wetness. No allowance was made for moisture in the 6-inch-to-3-inch grade, since the amount involved was found to be well below the degree of accuracy of the weigh-batcher.

Standard 6-inch test cubes were made at regular intervals from all concrete, samples of the concrete containing 6-inch stone being first wet-screened through a 2-inch-mesh screen. Standard slump tests were also made at the same time, but the harsh nature of the aggregate made them of doubtful value, results rarely registering outside the range of 1 to 2 inches.

In addition to the 6-inch cubes, a number of 18-inch-diameter by 3-foot-long cylinders were made from the Class D concrete both with the full

mix of 6-inch stone and from wet-screened samples, and these were tested on the 500-ton machine at University of Birmingham.

A few blocks of the Class D concrete were also cut away from the downstream temporary face of the dam, dressed into cubes, and similarly tested. It had been hoped to drill cores of 18 inches diameter or larger from the completed work, for testing, but the special plant necessary could not be located and the cost of adapting plant to obtain a few cores was felt to be prohibitive.

At the beginning of concreting, test cubes were made in sets of nine, three cubes being tested at each age of 7, 28, and 90 days. Later, because of the inconsistent results referred to earlier, tests at 28 days were discontinued and six cubes only were made thereafter, except in a few cases when a set was made for testing at 6 or 12 months. Table 5 shows typed test results indicating gain of strength with age.

The use of 6-inch aggregate in the concrete mix precluded the testing of 6-inch cubes of the actual concrete placed, and in order to ascertain the relationship between the wet-screened samples which were tested and the 6-inch concrete as placed, additional tests were carried out on 18-inch-diameter cylinders using screened and unscreened concrete.

TABLE 5

Class D low-heat-cement concrete	Strength gain with age				
	Compressive strength : lb. per sq. inch				
Test No.	7 days	28 days	90 days	6 months	12 months
547 (no admixture) . . .	842	1,898	3,563	—	4,895
820 (Teepol 410)	873	—	4,588	5,407	—
1138 do . . .	580	—	2,562	4,500	4,000
1318 (Teepol 530) . . .	768	—	3,443	3,983	4,748
1594 do . . .	1,080	—	2,965	4,645	5,123*
					* at 9 months

The results of these tests, which are given in Table 6, indicate that wet-screened concrete cylinders give a test strength equal to or more than 60 per cent of that of a 6-inch cube of similar material. This conforms to results achieved in the United States. Cylinders made from unscreened concrete gave strengths of up to 70 per cent of the screened 6-inch cube and it is fair, therefore, to assume that the screening of the concrete for testing purposes resulted in a somewhat lower test strength being credited to the concrete than it actually achieved.

Table 7 shows comparative strengths of 6-inch test cubes and cubes cut from the dam, and these indicate that, at 3 months, the test cubes

correspond fairly closely in strength to the concrete as placed, but there is little doubt that the concrete cut from the dam suffers to some extent in the cutting, and the cylinder test is considered to be the more reliable.

TABLE 6.—CLASS D LOW-HEAT CEMENT CONCRETE CYLINDERS

	Cylinder ref.	Age at test: months	Weight: lb. per cu. ft	Strength: lb. per sq. inch	Strength of standard 6-inch cube from same sample, at age 3 months: lb. per sq. in.
18-inch-diameter by 36-inch-long cylinders, full mix	1	3	148·3	2,720	3,820
	2	3	150·6	3,264	4,588
	3	3	148·8	2,315	3,660
	4	3	148·8	2,265	3,542
	5	3	149·8	1,800	3,308
	6	12	148·0	3,180	2,562
	7	3	151·5	1,750	3,982
	8	3	149·0	2,640	4,750
	12	6	150·6	2,815	2,707
18-inch-diameter by 36-inch-long cylinders, wet-screened	9	3	147·0	2,500	4,252
	10	3	145·0	2,160	3,493
	11	3	146·2	2,575	3,815
	13	6	151·8	3,640	2,965

TABLE 7.—CONCRETE BLOCKS CUT FROM THE STRUCTURE

Block ref.	Approx. size: inches	Age at test: months	Weight: lb. per cu. ft	Strength: lb. per sq. inch	Strength of standard 6-inch cube made from same sample, age 3 months: lb. per sq. in.
A	9	12	146·8	4,130	4,118
B	9	6	146·0	2,890	2,838
C	15½	3	143·5	1,885	3,372
D	15	9	154·0	3,355	3,470
E	14	7	151·0	3,370	3,295

CONCRETE TEMPERATURES

Electrical-resistance thermometers were embedded in selected positions (*Fig. 8* (*b*)) in the concrete of one leading block (No. 9) and the adjoining closer block (No. 8) to observe temperature gradients. Twelve thermometers with "Pyrotenax" leads were first obtained, but after seven of these had been embedded five proved to be unserviceable.

Fig. 2

Grouting Operations in Block 21 Cut-off Trench (Sept. 1948)

Fig. 4

View of Dam, facing East

VIEW

Fig. 6

EXCAVATING AND CONCRETING IN BLOCKS 1 TO 12 (AUG. 1949)

COMPLETED

Fig. 9

VIEW OF SITE LOOKING SOUTH-EAST (JUNE 1949)

Fig. 10

View of Site, looking South-East (June 1950)

Fig. 17

View facing West, showing Casting Bay for 40-foot Arch Ribs

The trouble was traced to slight dampness, in the hygroscopic magnesium-oxide insulation of the leads, which was sufficient to upset the small operating current of about 4 milliamperes. Five replacements with rubber-covered leads were obtained and fixed in alternative positions, and special care was taken to dry out and seal the remaining Pyrotenax leads before embedding them ; no further trouble arose, but some useful information was lost from this cause.

All the thermometers behaved at first in a similar manner, rising to a first peak of 60° F., and in 2 to 3 days after concreting to 70° F., falling 5° to 15° during the next 10 to 15 days. A second peak occurred after 3 to 4 months, followed by a gradual fall.

The maximum temperature recorded was 82° F. which occurred in two cases at the second peak, and the greatest temperature gradient between thermometers on one horizon at 1,065·00 O.D. was 34° F. between the centre and the upstream face of the closer block No. 8 with air temperature just about freezing point.

The thermometers near the upstream and downstream faces were influenced by the filling of the reservoir and by the building-on of the downstream masonry facework respectively, and reacted similarly by a rise in temperature of 8° to 10° F., thereby reducing the temperature gradient across the dam. The temperature of the centre of leading block No. 9 at level 1,065·00 O.D. was raised 8° F. when the concrete of the adjoining closer blocks Nos 8 and 10 was placed.

Readings were continued over a period of about 3 years and show that, in 12 months, internal temperatures had fallen to 60° F. and have now settled down at about 55° F. Thermometers near the faces and in the top thinner sections of the dam continued to be influenced by the air temperature.

Contraction Joints

Radial contraction joints were formed at 46-foot-6-inch intervals through the dam with vertical keyways at 20-foot intervals, the joints between blocks of the dam being specified to be made with greaseproof paper.

The contractor was permitted to fabricate climbing shutters (see *Fig. 6*) for this work from standard 4-foot-square pressed-steel tank plates which contained the usual embossed crosses on the surface (see *Fig. 2*) so that the finished concrete faces were not smooth, and it was decided to omit the greaseproof paper from the joints, the concrete faces of leading blocks being coated with mould oil immediately before placing concrete in the closer blocks.

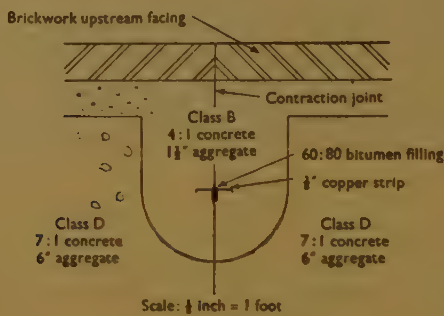
A continuous vertical water-stop (*Fig. 13*) was formed in each joint, about 6 feet from the upstream face, using ⅛-inch-thick annealed copper sheeting, 2 feet 9 inches wide, bent to U shape, successive strips being

butt-jointed and fully welded. The 4-to-1 Class B concrete backing the upstream face was returned locally along each joint to embed the copper sheet by at least 3 feet.

In view of the acid nature of the water, some anxiety was felt as to possible electrolytic action between the copper and the composite weld material which was a " Brazotectic " silicon bronze composed of 94 per cent copper, 4 per cent silica, and 2 per cent zinc.

Twelve sample strips of copper sheet containing welds were made up and weighed in a laboratory. These were then immersed in the river up-

Fig. 13



WATER STOP

stream of the site and samples were removed at intervals for re-weighing and microscopical examination.

Results from all the samples were very consistent. A loss on the original weight of 0·66 per cent took place over the first 8 months' immersion, and thereafter the rate of loss decreased, being 0·84 per cent after 12 months' and 1·5 per cent after 2 years' immersion.

Microscopical examination showed slight pitting of the weld material, the bottom of the pits being slightly rounded, suggesting that they were at least partially formed by a scouring action. There was also slight dezincification at the bottom of some of the pits, suggesting some corrosion attack as well.

It was considered that these samples were subjected to more severe conditions than would obtain in the case of the actual water-stops in the dam, but additional protection was given by filling the U on the upstream side with a 60/80-penetration bitumen poured hot just before placing the concrete to the closer blocks.

Since no method could be devised for testing the watertightness of the welds in situ, a length of copper sheeting in the work was selected at random and cut away. This was reduced to a sample about 4 inches long containing a weld, the cut edges were fettled-up and a half-round

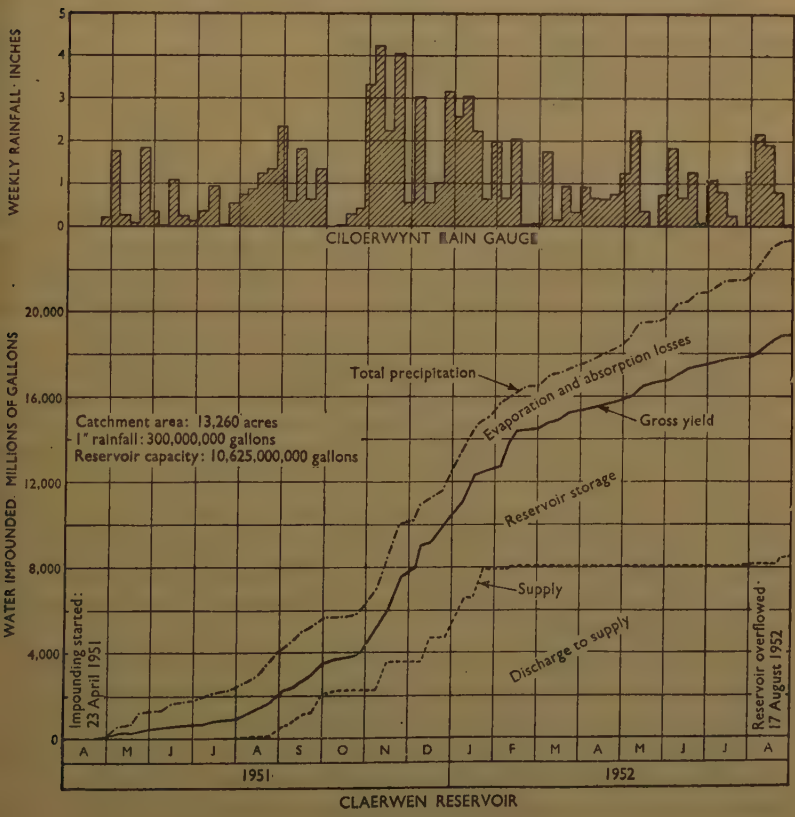
strip of copper was welded on so that the sample could be made up to form a side of a watertight box by clamping rubber-jointed steel plates on top and bottom.

This box was subjected to a hydraulic test which proved two pin-hole leaks in the weld at a pressure of 25 lb. per square inch. The box was dismantled and two coats of hot bitumen were painted over the weld area and the test repeated. No leakage was then apparent up to a pressure of 250 lb. per square inch—about $2\frac{1}{2}$ times the maximum working head—at which point the sample failed by buckling.

Impounding

In view of the shortage of water for supply being experienced towards the end of each summer, efforts were made to start impounding at the

Fig. 14



Rainfall and Run-off Curves

earliest possible moment. Unfortunately, owing to delay in obtaining delivery of all the necessary sluice valves, the bulk of the 1951 spring rainfall was lost, but impounding was started on 23rd April, 1951, when the level of the concrete in the lowest block was still 68 feet below crest level.

Discharge of water to supply had to be started on 31st July through the scour mains and was continued until the 13th October. Heavy autumn rainfall made it necessary to discharge further quantities in November and December 1951 and January 1952 to prevent the water level from interfering with the work.

The rainfall (*Fig. 14*) during the first 12 months of impounding was 60·4 inches measured at a gauge just downstream of the site, so that the total theoretical precipitation on the Claerwen catchment at 300 million gallons per inch of rain was about 18,120 million gallons.

The actual quantity discharged during the year was 8,000 million gallons, and the quantity stored in the reservoir at the end of the 12 months was 7,730 million gallons, so that absorption and evaporation losses were about 2,390 million gallons or 13·2 per cent.

After some subsequent discharge, the reservoir filled and overflowed on Sunday, 17th August, 1952.

Hydraulic Model Experiments

A series of experiments was carried out at the site on three hydraulic models, the primary object of which was to check the capacity of the spillweir arrangements and the behaviour of the jets issuing from the draw-off mains at the base of the dam, and to calibrate the measuring weir in the centre spillweir channel.

A suitable site was available in a small tributary of the Claerwen river near to the site offices where the models could be fixed with the minimum of costly preparation and be supplied with water by gravity. The site was not enclosed and the experiments were limited to periods of favourable weather.

The models were constructed in timber or thin tin-plate, masonry facing being simulated by cement plaster or grit set in varnish, and concrete surfaces reproduced by varnished wood. Glass panels were inserted in channel walls to observe turbulence, and a full series of photographs was taken of all the experiments.

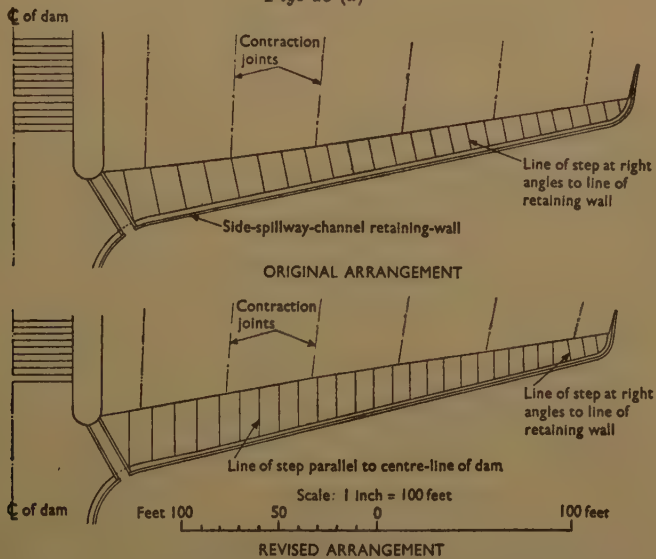
Model I, scale 1 : 48, comprised the full length of the spillweir crest and downstream face of the dam below it, together with the side and centre spillweir channels, weirs, footbridges, and draw-off main outlets (*Figs 15* (*a*)).

Model II, scale 1 : 8, was built to confirm certain conclusions drawn from experiments with the first model, and represented a 12-foot length of the central spillweir crest and 32 feet of downstream face below it (*Figs 15* (*b*)).

Model III, scale 1 : 18, represented the centre spillweir channel and weirs to half width for the purpose of calibrating the measuring weir.

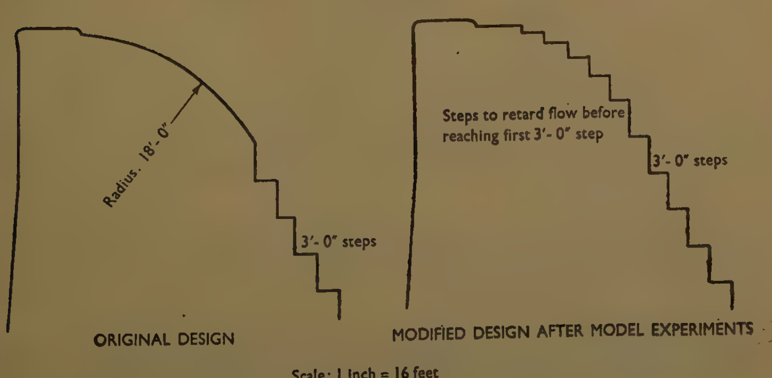
As a result of the experiments certain modifications were made to the angle of the steps in the floors of the side spillweir channels to improve the discharge, and the top of the central 60-foot stepped spillweir crest was re-designed (*Figs 15*) to overcome a tendency for a nappe to form at certain flows.

Figs 15 (*a*)



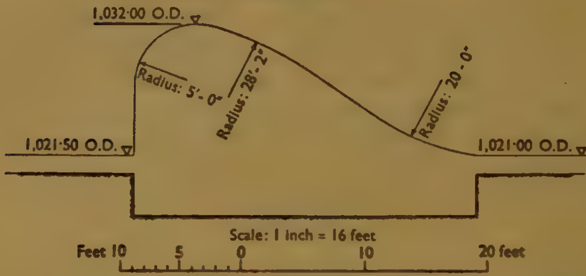
ORIGINAL AND REVISED ARRANGEMENTS OF SIDE-SPILLWAY STEPS

Figs 15 (*b*)



DESIGN OF STEPPED SPILLWAY CREST

Fig. 15 (*c*)



PROFILE OF MEASURING WEIR

Details of the tests are not given in this Paper, since they were applicable only to the particular problems mentioned, but reference is made to them because the subsequent behaviour of the re-designed crest and measuring weir has proved the accuracy of model-work of this description, carried out under site conditions. The dam has not yet overflowed to an extent sufficient to test the accuracy of the model-work done on the side spillweirs.

ROADWAY VIADUCT—PRESTRESSED CONCRETE ARCHES

The roadway viaduct, 22 feet wide overall, consisting of thirteen elliptical arches, the central one of 60 feet span and 12 feet rise, flanked on either side by six of 40 feet span and 8 feet rise, clearly lent itself to prefabrication ; at the Contractor's suggestion the original design in reinforced concrete for the twelve spans of 40 feet was replaced by one in prestressed concrete, in which each arch consisted of eight two-hinged members placed side by side, the size of each member being limited only by the lifting capacity of the cableways.

Later, owing to difficulty in obtaining normal reinforcing steel, the 60-foot span was also constructed in prestressed concrete, but this span was built in situ.

Prestress was obtained by post-tensioning Freyssinet cables consisting of twelve wires of 0·2 inch diameter encased in plastic sheaths. The design load was 50,000 lb. per cable, plus 15 per cent for creep and shrinkage in concrete and steel, and with an additional allowance for friction amounting in all to 10,000 lb.

For the 40-foot spans the two outer ribs faced with masonry voussoirs 9 inches thick, were each 2 feet wide and contained four cables. These ribs weighed 18 tons each and two cableways working together were required to handle them.

Between the two outer members in each arch were placed six ribs, 3-feet wide, each weighing about 9 tons and containing six cables. The end blocks for the ribs containing the anchorage cones and distribution

reinforcement cages were precast separately from the ribs using a slightly richer concrete. (See Figs 16, Plate 2.) Two casting beds for making 3-foot ribs (*Fig. 17*) and one casting bed for the outer ribs, capable of making sixteen ribs in all at a time were used, and altogether ninety-nine ribs were made. Of these, one was tested to destruction and two were rejected.

The concrete strengths specified were 5,700 and 7,500 lb. per square inch at 7 and 28 days respectively, and it was found necessary to use rapid-hardening cement with a water/cement ratio not exceeding 0·42 to meet this requirement. A cement content of 750 lb. per cubic yard for the rib concrete was increased to 1,000 lb. per cubic yard for the end blocks, owing to the dense reinforcement.

Since all this work was carried out between December and March, steam curing as a frost precaution was used during the colder weather for 18 to 24 hours after pouring the concrete, and this resulted also in accelerated hardening. A number of shrinkage cracks occurred in the first ribs cast, and thereafter one definite joint at the crown was formed in all the subsequent ribs.

A temperature of about 100° F. was maintained, with the result that the specified 7-day strength was usually achieved in 5 days, thus allowing for earlier prestressing. This may have involved some loss of 28-day strength, although the test cubes were probably subjected to less favourable curing conditions than the ribs themselves, owing to difficulty in controlling the initial temperature rise in the curing hut.

The average 6-inch-test-cube strengths are shown in Table 8.

TABLE 8

Class	Weight: lb. per cu. ft	Water/cement ratio	Compressive strength: lb. per sq. in.			
			5 days	7 days	14 days	28 days
X rib ends	147·5	0·39	—	6,830	7,745	8,715
Y ribs	151·2	0·42	—	6,400	7,730	8,595
Y ribs (steam cured)	..		6,425	—	6,560	7,410

Note: A number of individual cubes exceeded 10,000 lb. per square inch at 28 days.

Considerable difficulty arose at first in obtaining the necessary elongation, because of the friction, which was so great that a cable might have to be re-stressed two or three times with a 32-per-cent overload. The repeated

ramming of the cones, which indented the wires each time, caused a number to break and these could be replaced only with difficulty.

Various methods of overcoming the friction, such as lubrication of the cables and vibration during stressing, were tried, but without much success. Ultimately the Contractor devised a satisfactory method as follows :—

1. The cable was pulled to and fro about 6 inches to free it in the passage. This was done not less than 3 days after pouring when the concrete had hardened.
2. The cable was loaded to 71,500 lb. (85-tons-per-square-inch stress in the steel).
3. The jacks were then released, when the cones would hold the load at about 38,000 lb. (45 tons per square inch).
4. The cable was reloaded to the maximum of 79,100 lb. (94 tons per square inch).
5. This load was held for 5 minutes and the cones then rammed.
6. The cable passages were grouted and the bronze hinge-plates cast on.

To ensure adequate attachment of the masonry facing to the concrete of the outer members, the two end stones on each rib were dowelled horizontally to the concrete, and steel mesh—placed in each radial joint between stones—extended into the concrete behind.

When lifted, the ribs were set to level on copper shims on the springers, the gap between the bearing-plates on the ribs and the bearing-faces of the springers being filled with a dry 1-to-1 sand/cement mortar and well caulked up. After the eight ribs forming one arch had been set, they were covered by a 12-inch layer of Class B concrete, reinforced transversely to tie them together, and on this the spandrel walls were constructed.

It was estimated that a central point load of 15 tons on a 3-foot-wide rib would induce bending moments approximating to those of the actual dead and live loads, and the first rib to be made was tested to destruction to observe results. At 15 tons load the central deflexion was $\frac{1}{4}$ inch, and the loading was then increased by increments of 5 tons to $42\frac{1}{2}$ tons when one hinge failed by shearing, the deflexion then being about 1 inch. Had the hinges been restrained against horizontal movement, this rib would have withstood a greater load. Five more ribs were later tested up to a load of 20 tons, the central deflexion being very uniform at $\frac{1}{4}$ to $\frac{5}{16}$ inch.

The 60-foot-span arch, which was built in situ, had masonry voussoirs 24 and 18 inches thick alternately which were treated as part of the arch and drilled to take the cables. To avoid the trouble experienced with friction in the 40-foot spans, the design load was reduced to 40,000 lb. per cable by increasing the number of cables in the vault. Even so, in some cases the maximum jacking load was required.

It is thought that much of the difficulty in obtaining the prestress required was attributable to the plastic sheaths. The three surplus ribs

were later broken up for examination, and it was found that the passages were very irregular in places owing to the sheaths having rucked up, especially round the sharp bends near the ends of the ellipse. It is suggested therefore, that sheaths of tin tube or holes formed by the "Ductube" method would probably have proved less troublesome.

Survey of Reservoir Area

An aerial survey of the reservoir area to a scale of 1 : 2,500 with contours at vertical intervals of 5 feet was made at a cost of £670. The aerial photography was done on one day, in less than one hour, and was followed by about 10 days' work, by two surveyors in location of control points on the site, and by a few weeks' work in the survey firm's plotting laboratories.

The firm guaranteed 90 per cent of all levels to be accurate to within 2 feet and 95 per cent to within 2½ feet. Random ground checks proved the contours to be well within the guaranteed accuracy. The main purpose of this survey was to prepare the Table of Contents of the reservoir, and so far as is known this is the first occasion on which aerial survey has been used in Great Britain for this purpose.

Quantities and Costs

The quantities of the main items in round figures were as follows:

Excavation:	Soft	47,500 cubic yards
	Rock	127,000 cubic yards
Concrete		378,800 cubic yards
Brickwork, average 13½ inches thick		4,000 cubic yards
Masonry, average 10½ inches thick		7,800 cubic yards
Masonry, dressed ashlar		1,500 cubic yards
Total volume in works		392,100 cubic yards

The tender, which was accepted in May 1946, amounted to £1,494,440, and was subject to adjustment by reason of variations in cost of labour, permanent materials, and insurances. The final cost has not yet been ascertained but is expected to be in the region of £2 million.

Part 3.—Construction Plant and Methods

by

R. H. Falkiner, B.A., B.A.I., A.M.I.C.E.

Plant Employed

The general lay-out of the construction plant is shown in Fig. 18, Plate 2, and *Fig. 19* (facing p. 282).

The main requirement was a set of plant that could transport and place

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in position about 360,000 cubic yards of materials in a period of 3 years. The overall height of the dam is 250 feet and the plan area is about 3 acres. To cover this space by standard derrick cranes would require about nine machines mounted on a gantry 160 feet above the lowest ground level, and the expense of such a structure and the delay in its installation led to its rejection.

On the other hand, the section of the valley, with hillsides rising fairly steeply above the top level of the dam, was suitable for the use of cableways, and these were adopted.

The cableways were manufactured specially for the job by Messrs John Henderson of Aberdeen. Two were provided, each capable of a 10-ton load, with a travelling speed of 1,000 feet per minute and a hoisting speed of 250 feet per minute. The head-masts were fixed and the tail-masts travelled on an arc track 400 feet long which enabled the whole of the dam to be covered. The cableways were electrically driven and all motions controlled from the head-mast end. Since it was impossible to arrange for drivers to see their loads at all times, signals were given by telephone and remarkably good control was obtained.

One disadvantage of cableways is that the very fact of their range, speed, and power makes it uneconomical to use them for small miscellaneous lifts, for example, raising shutters, setting masonry, etc. This was overcome by providing mobile diesel cranes and electric rail cranes of up to 3 tons capacity. These were lifted and transported about the job by the cableways.

Power Supply

The nearest point on the public electricity supply was 25 miles to the east, and the price quoted for installing power lines to Claerwen was uneconomical compared with diesel-driven plant at the site. The capacity of the plant provided was 1,100 kilowatts, made up by two Mirrlees, Bickerton & Day sets of 325 kilowatts and two English Electric sets of 225 kilowatts.

These were later supplemented by a 300-kilowatt Ricardo Paxman set to operate only when another unit was undergoing overhaul. Power was generated at the standard 440-volt 3-phase supply and distributed to the consumption points by overhead conductors. In all, nearly 4 million units were generated, and failures of the supply were considerably less frequent than would have been experienced on a public supply during the particular period.

Cement and Concrete Plant

It was arranged that cement should be delivered in bulk containers of 3½ tons capacity. These were provided by the Great Western Railway

who delivered them to site from South Wales. Containers were hauled three at a time in a rail truck to Rhayader and two at a time on a lorry to site. At site they were off-loaded by 5-ton overhead travelling hoists and emptied through the bottom door into one of three cylindrical silos each holding 150 tons.

From the bins the cement was fed through 10-inch-diameter Saunders valves to a Redler chain-conveyor which delivered it to the cement-bin above the concrete plant.

The concrete plant consisted of the bins referred to, having a total capacity of 120 tons, weigh-batchers dealing with all materials including water, a 3-cubic-yard-capacity tilting-drum Winget Koehring mixer, and a wet-hopper of 7 cubic yards capacity.

This was set up on a reinforced-concrete trestle so that concrete could be fed through a breeches chute to 4-cubic-yard-capacity roller-bottom skips mounted on purpose-made broad-gauge bogies. The bogies, of which there were two running on separate tracks, were each large enough to accommodate two skips. They were shuttled to the cableway pick-up by capstan-type winches, where the incoming empty skip was landed and unhooked and the full skip hooked on and hoisted away.

Miscellaneous Plant

For the excavation for foundations, three excavators of $1\frac{1}{4}$, $\frac{7}{8}$, and $\frac{5}{8}$ cubic yards respectively were used, and these were served by a fleet of twelve $4\frac{1}{2}$-cubic-yard diesel-driven dumpers. Compressors of 1,500 cubic feet per minute capacity were installed in the power station.

Quarry

The nearest site for a workable quarry was at a point 4 miles from the site of the dam and more than one mile from the nearest road. It was not practicable to transmit power from the main site, so it was necessary to carry the lump stone from the quarry (*Fig. 20*, facing p. 283) to a crushing plant at the main site. Special lorries with rock bodies were provided for this duty.

The choice of the quarry site at Marchnant was dictated by the existence there of an outcrop of gritstone in a bed about 90 feet thick. There are in the area many beds of this type of stone, but they are as a rule only about 20 feet thick and usually unworkable for any large quantity.

The stone at Marchnant was an extremely hard silicious grit containing about 90 per cent silica; it was mainly massively bedded with a dip of about 20 degrees striking toward the quarry face. The top surface of the rock was deeply serrated and the overburden of glacial drift ranged from nil to 10 feet; the problem of clearing this was never adequately solved.

As a result, much overburden became mixed with the stone on the floor and considerable amounts of good stone had to be rejected on this account. About 23,000 tons were recovered from the waste-dumps by passing it over a vibrating-bar grisly which removed the smaller stones and most of the dirt. In addition to the dirt arising from the overburden the quarry was marred by considerable joints filled with clay, which necessitated washing of the stone. This was done partly in the lorries at the quarry and partly in the chute above the crushers. Some difficulty was experienced because of seams of shale in the quarry and it was necessary to load a certain amount of stone by hand to ensure the rejection of the shale. As the quarry developed, it was found possible generally to avoid the shale and to obtain the whole output from the more favourable strata.

The gritstone was so hard and abrasive that drilling was a major difficulty, which was overcome by the use of Hydralloy tungsten-carbide bits with 3½-inch drifter drills mounted on wagons. With these, 20-foot holes, 2⅝ inches in diameter, were drilled horizontally at quarry-floor level, and this was the main pattern of drilling. Where the face was too high for this to be effective, raking holes, up to 30 feet deep, were drilled in addition. The biggest blast brought down more than 25,000 tons of stone, and the overall consumption of explosive was ⅛ lb. per ton.

Lump stone was loaded into lorries by shovel excavators, two of 1¼ cubic yard capacity and one of 2½ cubic yards. The maximum outputs were 1,000 tons per day and 4,500 tons in one week. On delivery to the site, the stone was tipped into a long chute and was fed from there by plate-apron feeders into the primary crushers.

A stockpile of each size of aggregate was deposited over a buried conveyor which collected each size in turn and delivered to the stock bins over the concrete plant.

Originally it had been proposed that the aggregate should consist of crusher-run material from 6 inches down with an addition of smaller sizes to correct the grading. In practice it was found that the grading of the crusher-run material varied so much from time to time that correction was virtually impossible. The variation arose partly because of the quality of the stone and the size fed to the crushers; but the main cause was segregation of the sizes as the material was placed in and taken out of the stockpile.

Excavation of Dam Foundations

Excavation was almost entirely in mudstone, on which the dam is founded. Generally this material is similar to slate but with less perfect cleavage and more strongly marked beds and joints. Initially it was drilled vertically using jackhammer drills and ordinary forged drill-steels. Owing to the nearly vertical cleavage, blasting results were very poor, and horizontal drilling was introduced as soon as was practicable. Thereafter drilling and blasting presented no problem.

Fig. 19

WORK IN PROGRESS, AUGUST 1951

Fig. 20

VIEW OF QUARRY SHOWING THREE-LEVEL METHOD OF EXCAVATION

Practically the whole of the excavated material was loaded direct into dumpers which worked on a series of ramps to the tips and were located upstream of the dam. Altogether, 174,500 cubic yards were excavated. Progress at the average rate of 2,000 cubic yards per week was maintained over long periods.

The first work undertaken was to form the stream-diversion channel which was cut in the rock on the east side of the river, clear of the three central blocks of the dam. This channel was 820 feet long and 8 feet wide at the base with sides formed at 30 degrees to the vertical. Its greatest depth was more than 25 feet. The work was done by dragline, which made only slow progress because of the bad fragmentation of the rock. It was necessary to clear off a lot of badly broken rock from the top of the low side of the channel and then to construct a concrete training wall for most of the length of the channel. Later, when excavation was carried down below water level, it was found that the rock in the sides and bed of the channel was badly cracked, causing an undue inflow of water. This was successfully dealt with by cement grouting.

The stream was diverted during a period of dry weather by a temporary dam of clay bags, and the diversion dams upstream and downstream of the main structure were constructed. Each of these consisted of a concrete core founded on rock with soft fill covered with boulders on both sides. The cofferdam so formed was flooded on three occasions by exceptional rainfall, but no damage was done and it was necessary only to pump out the overflow.

After the diversion of the stream, excavation for the central blocks Nos 10, 11, and 12 was carried out, an 8-foot-square sump being sunk 55 feet below river-bed level or 10 feet below deepest excavation level. Pumping was always within the capacity of a 6-inch sinking pump.

Cut-off trenches were excavated, so far as was practicable, with a small drag-shovel to depths of 8 to 10 feet, but the remainder down to 15 feet below general formation level was loaded by hand into rock trays which were hoisted away by cableways.

Having cleared the bulk excavation by machine, a very considerable amount of work remained in clearing out all loose or doubtful rock from the exposed surfaces; this could only be done by hand, assisted by water jets.

Drilling and Grouting in Foundations

The grouting holes, which were $1\frac{1}{2}$ inch diameter, were drilled by C.P.5 rotary diamond drilling machines driven by compressed air. In general, non-coring-type drills were used and proved more satisfactory than core-type drills. The mudstone rock was not abrasive, so wear on the drill rods was negligible and breakages did not occur. In this lies a great advantage over percussion-type drilling.

Grout was pumped in by a reciprocating air-driven pump at pressures up to 250 lb. per square inch.

Concreting

It had been intended that the sand for concreting should be produced from the crushed stone. In fact, however, the concrete produced in that way was so unworkable that there was no prospect of proper compaction, and arrangements were made to import natural pit sand, of which 192,000 tons were brought by road from the Severn Valley. Even with this, the angular and rough fracture of the stones forming the coarse aggregate made a concrete difficult to compact. Large high-speed vibrators were used for this purpose and passable concrete of 7-to-1 strength was obtained.

The method of concreting is evident from the plant lay-out: the cableways proved a very convenient and efficient method of transporting and placing concrete. The labour involved was, on the average, about ½ man/hour per cubic yard for mixing, transport, and placing. This did not include preparatory cleaning or subsequent covering up and curing.

Outputs obtained were as follows:

Best shift	1,200	cubic	yards
,, week	4,225	,,	,,
,, month (5 weeks)	18,000	,,	,,
,, year (ending 25th April, 1951)	164,000	,,	,,

Formwork

Since nearly all exposed surfaces are clothed either in brickwork or random rockfaced squared masonry, the formwork was confined to construction joints. On account of the shortage of steel plate and timber, it was decided to use pressed-steel tank plates obtained *ex* Government stocks; these were assembled in 16-foot-by-4-foot panels and used as cantilever climbing shutters. A concrete screw was developed for securing these to the previous lift of concrete, and in this way the need for leaving in a steel washer-nut was avoided. The screws which were similar to giant woodscrews, 12 inches long by 2 inches diameter at the butt, are completely recoverable and can be used countless times.

The only intricate formwork was in the inspection galleries which involved two long stairways. The steps and part of the sidewalls were precast in blocks of seven steps for half a flight. These blocks were set on the mass concrete and concreted in, the walls and semi-circular roof being formed by timber shutters. In this way, a great deal of in-situ formwork was saved.

Masonry

It was originally intended that the masonry would be built up ahead of the concrete in 4-foot lifts but, owing to shortage of stone and masons, this had to be abandoned, and concreting was carried up behind rough shutters, a series of benches being formed so that cranes could be installed to handle and fix the masonry. Near the top of the dam, masonry was pushed up ahead of the concrete and the work was finished above level 1,186·00 in the way originally intended for the whole job.

In all, 26,800 square yards of facing masonry were fixed at a speed of about 500 square yards per week average over long periods. In addition, there is more than 40,000 cubic feet of ashlar masonry in arches, parapets, and string courses.

With regard to the organization of the random work, each course was designed by an engineer and the foreman mason; the stones required were picked out in the stock-yard and delivered in batches of about 6 or 8 tons by the cableway to the masons; in this way, it was possible to keep the working places free of unwanted material and nearly to eliminate cutting and dressing there. The largest stones used were approximately 4 feet by 3 feet by 18 inches, and weighed about 30 cwt.

Labour and Accommodation

The estimated peak labour requirement was 350 men. It was thought that about half would be found locally within daily travelling distance. It was decided, therefore, to construct a works camp to accommodate 200 men; this proved to be just adequate. Huts, accommodating 16 to 20 men, were provided with central heating from separate boiler houses, and normal messing and ablution facilities were provided. A recreation hall containing billiards room, bar, and cinema hall was built and a recreation club was registered. Sewerage works were built and electricity supply was generated at the site by diesel-driven dynamos delivering 100-volt D.C. supply to storage batteries of 900 ampere-hours capacity.

Technical and clerical staff were largely accommodated in twelve semi-detached bungalows built in traditional style. In this way a lot of wasteful expenditure on permanent services for temporary purposes was avoided.

Main Items of Plant Employed

Electricity Supply

2 Mirrlees, Bickerton & Day diesel generators, each of 325 kilowatts

2 English Electric supercharged diesel generators, each of 225 kilowatts

1 Ricardo Paxman diesel generator, 300 kilowatts

Total capacity: 1,400 kilowatts

Cableways

2 10-ton Henderson radial cableways

Crushing Plant

2 Broadbent 42-inch-by-36-inch jaw crushers
2 Parker 30-inch-by-18-inch jaw crushers
2 Parker 24-inch-by-12-inch jaw crushers
2 Parker Kubit No. 4. impact crushers
1 48-inch Gyrasphere crusher
1 6-foot-diameter rotary screen
1 5-foot-diameter rotary screen
1 12-foot-by-5-foot vibrating screen
2 Parker plate-apron feeders
Conveyors, elevators, bins, and chutes to feed the above units and to transport material to the concrete plant.

Concrete Plant

3-cubic-yard Winget Koehring Tilting Drum Mixer complete with weigh-batchers
3 cement-bins, of 450 tons total capacity, with unloading hoists and conveyors to mixing plant

Excavating Plant

1	Bucyrus Erie	54	$2\frac{1}{2}$	cubic yards capacity
2	Ruston Bucyrus	37	$1\frac{1}{2}$	,, ,,
1	,, ,,	33	$1\frac{1}{4}$	,, ,,
1	,, ,,	24	$\frac{7}{8}$	,, ,,
1	,, ,,	19	$\frac{5}{8}$	,, ,,
1	Caterpillar bulldozer	D.8.		
1	,, ,,	D.4.		
12	$4\frac{1}{2}$-cubic-yard Aveling Barford dumpers			

Drilling Plant

8 C.P. 50 drifter drills
4 wagon drill mountings
20 JA 45 jackhammer drills
2 CP5 diamond drills

Diesel-driven Compressors

2	215-cubic-foot	C.P. Portable
1	500 ,,	C.P. Portable
2	180 ,,	Portable
1	500 ,,	Robey fixed
1	400 ,,	B. & W. fixed

Electric Compressors

2 CP
2 IR

Transport

11 F.W.D. 10-ton-capacity trucks
4 Foden diesel 10-ton-capacity trucks
2 Thornycroft Amazon wagons, 8 tons capacity
2 Austin trucks
2 Bedford 5-ton trucks

Cranes

1 Smith 5-ton steam crane
3 Smith 3-ton electric crane
1 Neal 2-ton mobile diesel crane
1 Coles 5-ton mobile diesel crane
1 Jones 3-ton mobile diesel crane
1 Ransome No. 4 mobile diesel crane
1 Henderson 10-ton electric derrick crane
2 Butters 7-ton electric derrick crane
1 Butters 5-ton steam derrick crane

Acknowledgements

The dam was constructed for the City of Birmingham Water Department, for whom Sir William Halcrow and Partners were the Engineers. The Contractors were Edmund Nuttall, Sons & Co. (London), Ltd.

The Authors acknowledge with thanks the permission of Mr A. E. Fordham, General Manager and Secretary of the City of Birmingham Water Department, to write this Paper and to reproduce photographs and slides from the progress photographs of the work and the shortened version of the progress film which was made from the original film prepared for the Water Department.

Acknowledgements are made also to the following :—

Mr C. A. Risbridger, M.I.C.E., Chief Engineer, City of Birmingham Water Department, for information on reservoir capacity, etc.

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Mr V. H. Gritton, A.M.I.C.E., Chief Assistant Engineer and later Acting Resident Engineer, Claerwen Dam construction, for assistance at all stages in the preparation of the Paper.

City of Birmingham Industrial Research Laboratories, Mr R. H. Harry Stanger, A.M.I.C.E., Department of Civil Engineering, University of Birmingham, and the Fulmer Research Institute Ltd, for data and advice on testing of materials.

Leonard Ellis (16 M.M. Films) Ltd, for help and advice in the preparation of the ciné-film record of the construction.

The Paper is accompanied by fourteen photographs and ten sheets of drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared; and by the following Appendix.

APPENDIX

Design of Dam to give a Constant Maximum Stress

The method of design described below would appear to be more rational for high gravity dams than the "middle-third rule."

A. Considering first the purely theoretical case in which there is no hydrostatic uplift in the joints and cracks, it is assumed that :—

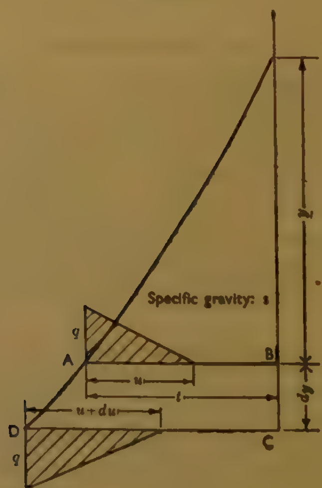
1. The distribution of stress is linear.
2. There is no tension.
3. The upstream face is vertical.

If q denote the stress in terms of feet head of water, then, using the notation given in *Fig. 21* for the horizontal element ABCD :

$$\tfrac{1}{2}qu + \left(t + \frac{dt}{2}\right)dy \,.\, s = \tfrac{1}{2}q(u + du)$$

$$\therefore\ st \,.\, dy = \tfrac{1}{2}q \,.\, du \qquad \ldots\ldots\ldots \quad (1)$$

Fig. 21



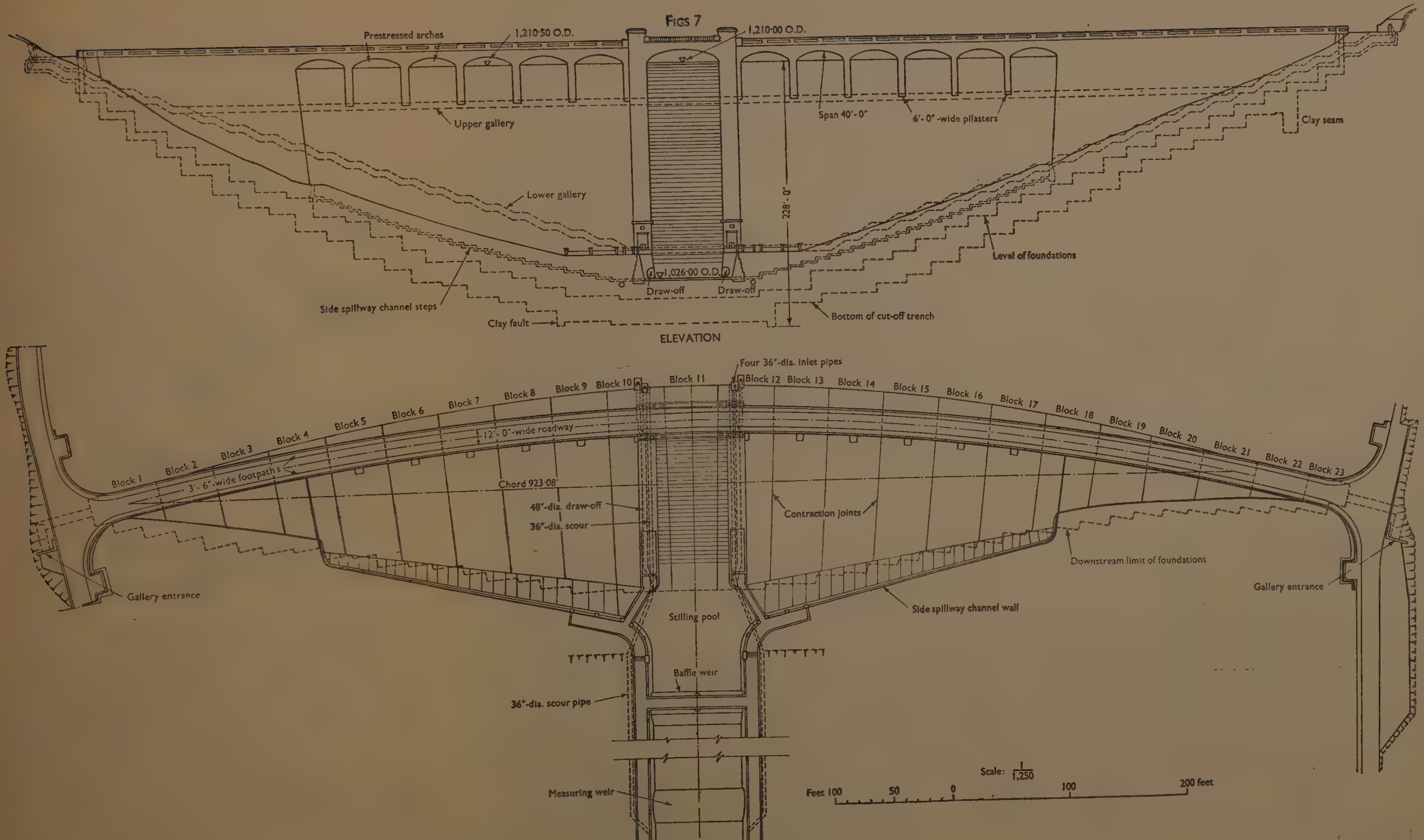
Taking moments about C :

$$\frac{y}{2}(dy)^2 + \tfrac{1}{2}qu\left(t - \frac{u}{3}\right) + \tfrac{1}{2}y^2 \,.\, dy + \tfrac{1}{2}\left(t + \frac{dt}{2}\right)^2 dy \,.\, s$$

$$= \tfrac{1}{2}q(u + du)\left(t + dt - \frac{u}{3} - \frac{du}{3}\right)$$

$$\therefore \qquad (y^2 + st^2)dy = qu \,.\, dt + q\left(t - \frac{2}{3}u\right)du \qquad \ldots \quad (2)$$

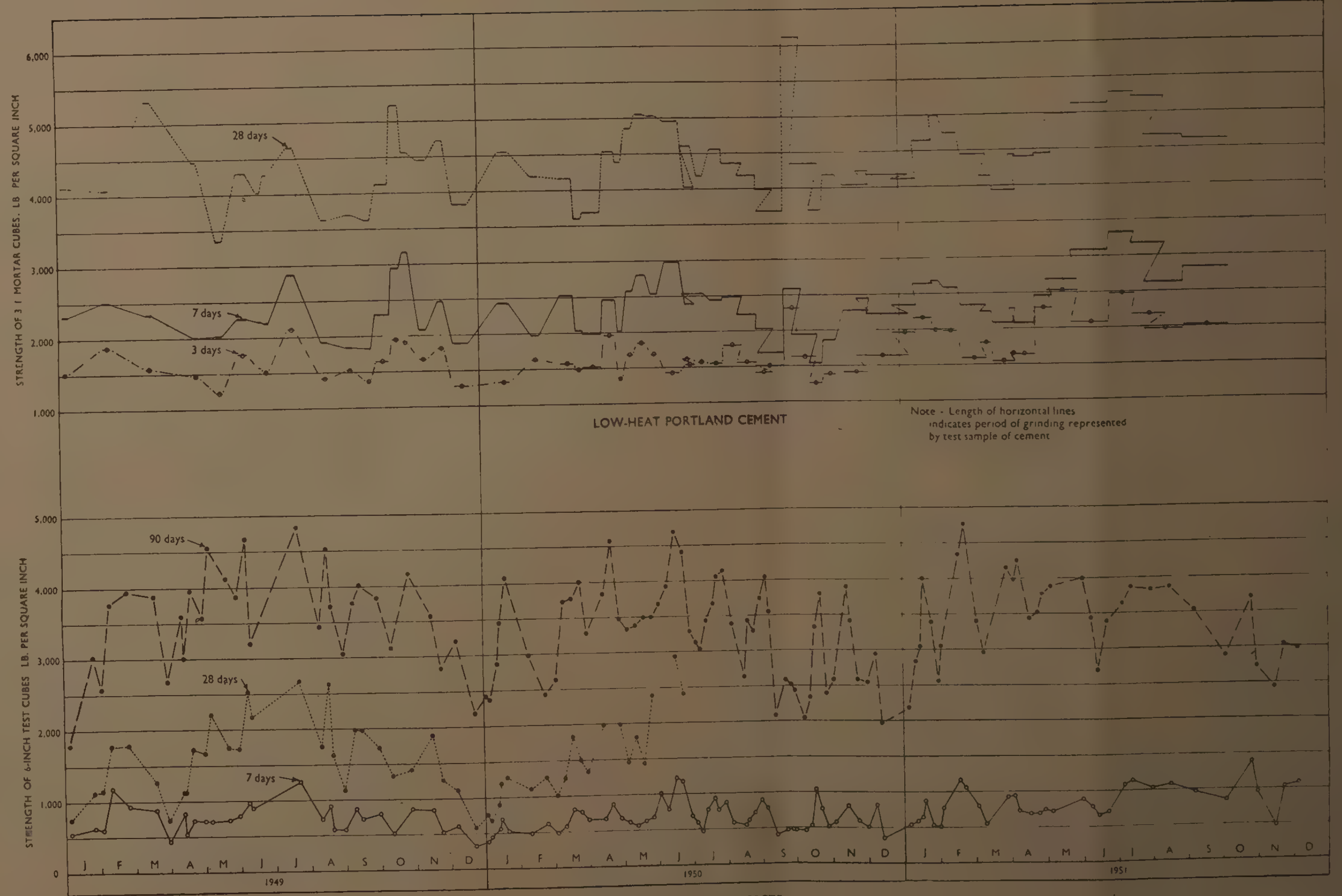
FIGS 7



PLAN

GENERAL ARRANGEMENT

FIG 12



CLASS D MASS CONCRETE

CONCRETE STRENGTHS

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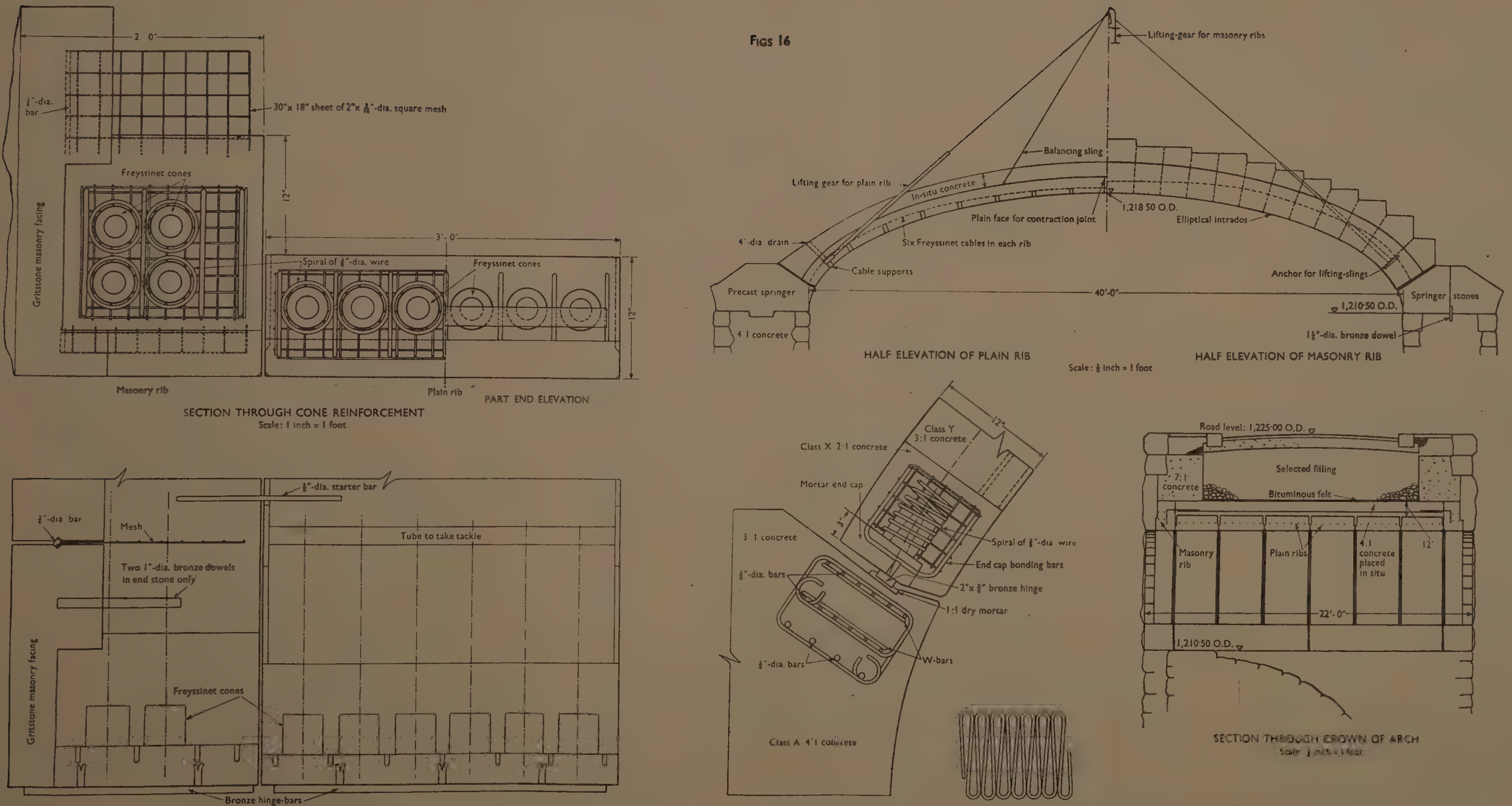
PLATE 2
CLAERWEN DAM

THE CLAERWEN DAM

Figs 16



SECTION THROUGH CONE REINFORCEMENT
Scale: 1 inch = 1 foot

PART END ELEVATION

HALF ELEVATION OF PLAIN RIB

HALF ELEVATION OF MASONRY RIB

Scale: ⅛ inch = 1 foot

PLAN ON TOP OF RIB ENDS
Scale: 1 inch = 1 foot

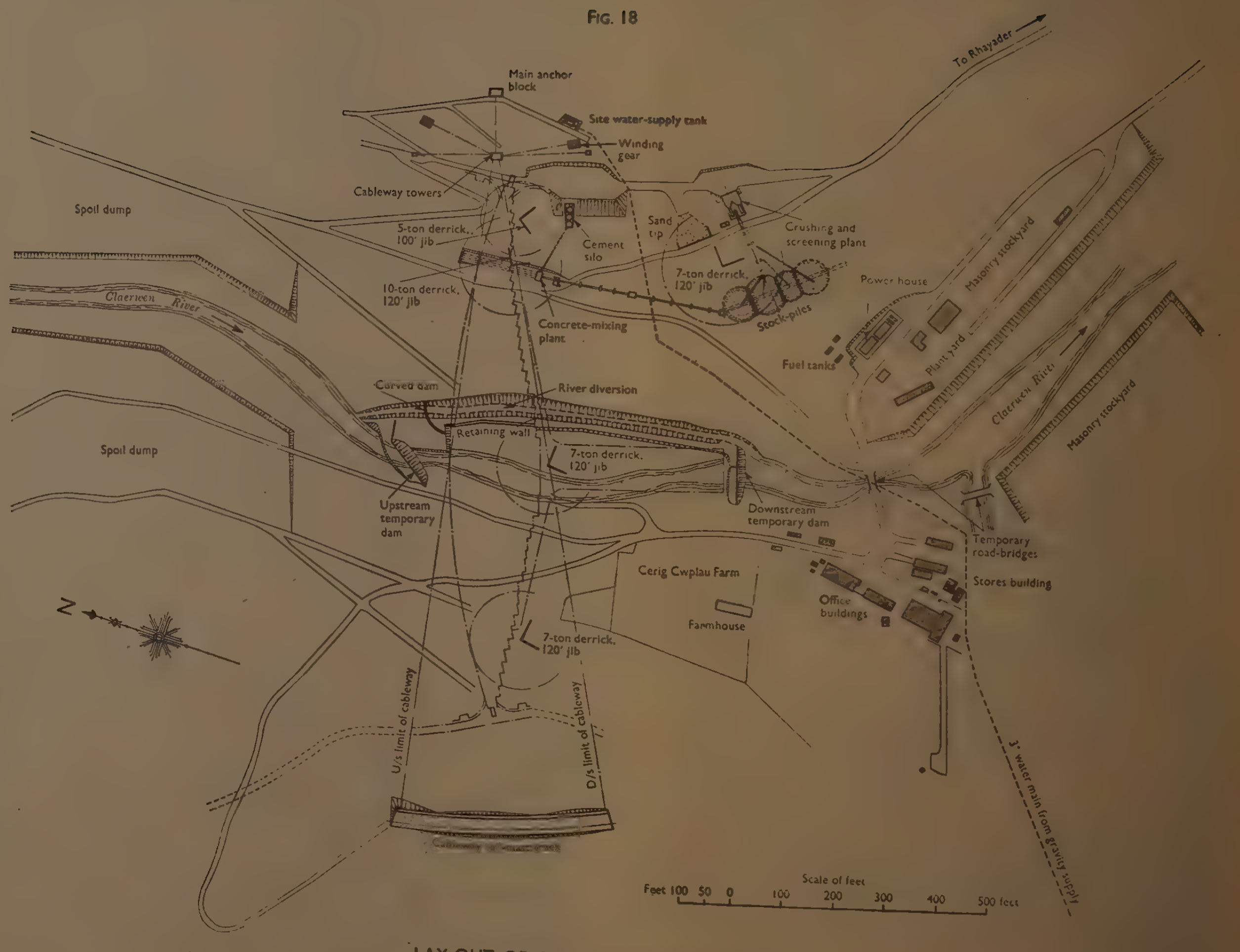
DETAIL AT HINGE
Scale: 1 inch = 1 foot

W-BAR

SECTION THROUGH CROWN OF ARCH

DETAILS OF PRESTRESSED ARCHES

Fig. 18



LAY-OUT OF CONSTRUCTIONAL PLANT

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William Clowes & Sons, Limited: London

From (1) and (2) $$(y^2 + st^2)dy = qu\,.\,dt + 2st^2\,.\,dy - \frac{4}{3}ust\,.\,dy$$

$$\therefore \qquad (y^2 - st^2)dy = u\left(q\,.\,dt - \frac{4}{3}st\,.\,dy\right) \quad \ldots \ldots \quad (3)$$

Putting $\gamma = \frac{2s}{q}\,.\,y$

equation (1) becomes : $$t\,.\,d\gamma = du \quad \ldots \ldots \ldots \ldots \quad (1)$$

and equation (3) becomes : $$\left(\frac{q^2}{8s^3}\gamma^2 - \frac{t^2}{2}\right)d\gamma = u\left(dt - \frac{2}{3}t\,.\,d\gamma\right) \quad \ldots \ldots \quad (3)$$

Writing $t = A\gamma + B\gamma^2 + C\gamma^3 + D\gamma^4 + E\gamma^5 + F\gamma^6 + \quad \ldots \ldots \quad (4)$

$$\text{Then } u = \int t\,.\,d\gamma = \frac{A}{2}\gamma^2 + \frac{B}{3}\gamma^3 + \frac{C}{4}\gamma^4 + \frac{D}{5}\gamma^5 + \frac{E}{6}\gamma^6 + \quad \ldots \ldots \quad (5)$$

$$\frac{dt}{d\gamma} = A + 2B\gamma + 3C\gamma^2 + 4D\gamma^3 + 5E\gamma^4 + \quad \ldots \ldots \quad (6)$$

$$\therefore \qquad \frac{dt}{d\gamma} - \frac{2}{3}t = A + \left(2B - \frac{2}{3}A\right)\gamma + \left(3C - \frac{2}{3}B\right)\gamma^2 + \left(4D - \frac{2}{3}C\right)\gamma^3 + \left(5E - \frac{2}{3}D\right)\gamma^4$$

$$\therefore \qquad u\left(\frac{dt}{d\gamma} - \frac{2}{3}t\right) = \frac{A^2}{2}\gamma^2 + \left(AB - \frac{A^2}{3} + \frac{AB}{3}\right)\gamma^3 + \left(\frac{3AC}{2} - \frac{AB}{3} + \frac{2B^2}{3} - \frac{2AB}{9} + \frac{AC}{4}\right)\gamma^4$$

and $$\frac{t^2}{2} - \frac{q^2}{8s^3}\,.\,\gamma^2 = \left(\frac{A^2}{2} - \frac{q^2}{8s^3}\right)\gamma^2 + AB\gamma^3 + \left(\frac{B^2}{2} + AC\right)\gamma^4 + \ldots$$

Adding together and equating each term to zero gives :

$$A^2 - \frac{q^2}{8s^3} = 0$$

$$7AB - A^2 = 0$$

$$\frac{11}{4}AC - \frac{5}{9}AB + \frac{7}{6}B^2 = 0$$

$$\frac{16}{5}AD - \frac{AC}{2} + \frac{5}{2}BC - \frac{2}{9}B^2 = 0$$

$$\frac{11}{3}AE - \frac{7}{15}AD + \frac{41}{15}BD - \frac{7}{18}BC + \frac{5}{4}C^2 = 0$$

$$\frac{29}{7}AF - \frac{4}{9}AE + 3BE - \frac{16}{45}BD + \frac{13}{5}CD - \frac{C^2}{6} = 0$$

The above equations give, in turn, by substitution :

$$A = \frac{q}{2s\sqrt{2s}}$$

$$B = \frac{A}{7}$$

$$C = \frac{2A}{99} = \frac{A}{49{\cdot}5}$$

$$D = \frac{5A}{4 \times 7 \times 7 \times 11} = \frac{5A}{2,156} \simeq \frac{A}{431}$$

$$E = \frac{2 \times 13 \times 34A}{11 \times 7 \times 99 \times 11 \times 49} \simeq \frac{A}{4,648}$$

$$\text{Hence } t = \frac{y}{\sqrt{2s}}\left(1 + \frac{\gamma}{7} + \frac{\gamma^2}{49{\cdot}5} + \frac{\gamma^3}{431} + \frac{\gamma^4}{4,648} + \cdot \cdot \cdot \cdot\right) \quad . \; . \; . \; . \; . \; . \quad (7)$$

$$\text{where } \gamma = \frac{2s}{q} \cdot y$$

B. Allowance for hydrostatic pressure, as for a pervious dam. If it be assumed that the hydrostatic uplift ranges from a maximum at the upstream face to zero at the downstream face, the effect is almost the same as taking the weight of the dam in water. This assumption is correct for a purely triangular dam and is a reasonable approximation in this case. Therefore it is necessary only to write $(s - 1)$ instead of s in formula (7), which gives

$$t = \frac{y}{\sqrt{2(s-1)}}\left(1 + \frac{\gamma}{7} + \frac{\gamma^2}{49{\cdot}5} + \frac{\gamma^3}{431} + \frac{\gamma^4}{4,648} + \cdot \cdot \cdot\right)$$

$$\text{where } \gamma = \frac{2(s-1)}{q} \cdot y$$

Discussion

The Authors introduced the Paper with aid of a film.

Mr C. A. Risbridger observed that the present very valuable Paper, dealing with the development of the Claerwen gathering ground, when added to the classic Paper presented in 1912 by the Mansergh brothers,[1] dealing with the development of the Elan gathering ground, would almost complete the thrilling story of the development of those gathering grounds. He said " almost " because the story might be rounded off in due course by information based upon the very careful records of rainfall, run-off, losses, and so on, which had been maintained on those gathering grounds during the past 50 years.

Although the present Paper dealt mainly with constructional matters, it would probably not be out of place to make very brief reference to the total storage which now existed in terms of the yield expected to be obtainable from the gathering ground. The number of days was in fact slightly more than 200, on the basis of a yield of 104 million gallons per day. That figure of 200 might be compared with a figure of 180, which Mansergh had intended to provide in his original project, and with 150 days, which would be obtained by applying the old Hawksley formula of $y = 500$ divided by the cube root of the average annual yield of the three driest consecutive years. It was a great tribute to Mansergh that with the information provided by only one long-standing rain-gauge on the gathering ground 60 years ago, supplemented by two or three very short-term gauges, he had

[1] See footnote 1, p. 249.

arrived at a guaranteed yield of 102 million gallons per day, which could not be seriously disputed today, when the additional records of 60 years were available.

During the passage of the 1940 Bill through the Select Committee, however, the petitioners had been able to convince that Committee that the yield would be at least 104 million gallons, and they therefore assessed the proper compensation now to be 29 million gallons per day instead of 27 million. One could only hope that that yield of 104 million would prove to be correct, but it had to be admitted that he would be a very brave man who would predict the yield of a gathering ground of that nature to within 2 million gallons out of 102 or 104 million, especially when it was considered that a wrong estimate of 1 inch in the average rainfall or of 1 inch in the expected losses from evaporation and absorption more than covered 2 million gallons per day.

Whilst on that subject, Mr Risbridger wished to call attention to some figures given in the Paper relating to the rainfall, the run-off, and the losses which occurred during the filling of the reservoir, and to say at once that they should not be taken seriously, for the following reasons. First, the rainfall had been taken from the record of one rain-gauge below the gathering ground, at an altitude of less than 1,000 feet, whereas the gathering ground itself, of more than 13,000 acres, rose to nearly 2,000 feet. In point of fact, determined from the nine rain-gauges which were maintained on the actual gathering ground, the rainfall figure was 67·4 inches, as compared with the figure of 60·4 inches given in the Paper. The matter might be thought to be unimportant, but he pointed it out just in case, in any possible future negotiations concerning that gathering ground or others, someone took up the figure of losses given in the Paper as 13·2 per cent of the rainfall of the 12 months. Actually the average annual losses over the past 50 years had just exceeded 21 inches, which was more than 30 per cent of the rainfall, and there had never been a loss by evaporation and absorption of less than 22 per cent, so that the figure of 13 per cent should be disregarded.

In order to complete the history of the development of the Claerwen gathering ground, it should be stated that the original project of Mansergh to provide three reservoirs in the Claerwen gathering ground had been given up on the advice of Mr Risbridger's last predecessor but one, Mr J. W. Wilkinson, who in consultation with Mr W. F. H. Creber had sought out a suitable site in the valley and had come to the conclusion that the one on which the dam had now been built was the correct one. Further, the hydrological calculations leading to a decision on the correct capacity, and therefore to the leading dimensions of the dam, had been carried out by Mr Risbridger's immediate predecessor, Mr Barnes.

If the Authors were given the opportunity—which no doubt every engineer at some time had wished that he could have—to do the job again, would they adhere to the same discharge arrangements? The

circumstances in the case in question, as had been briefly mentioned by Mr Morgan, were somewhat unusual, since in general, when water was wanted from the reservoir, not more than 50 million gallons per day would be required, and that raised the old problem of trying to discharge a small quantity of water under a high head. The Authors took it through a 48-inch needle valve, and with 50 million gallons per day and a 184-foot head the annular space through which that water had to come was very small, so that Mr Risbridger believed that there might be some trouble with cavitation.

He suggested for future consideration that one might adhere to the same arrangements as far as the 48-inch draw-off pipes, but instead of the needle valve at the end there might be a large cylinder—arranged at right angles to the main draw-off pipe in T shape— with various short branches of different sizes, each controlled by its own sluice valve, so that whatever quantity it was desired to get out of the reservoir would always be taken through a fully-open valve or valves. It was the throttling of valves which led to trouble. More accurate regulation of the flow through those fully open valves could be obtained by a secondary throttling of the downstream valve of the pair of guard valves in the valve chamber.

Mr R. C. S. Walters said that the use of vibrated concrete was very interesting. He and Mr Walton had used that for a small curved dam at Falmouth which has been a sort of military secret during World War II, and they had found that vibrated concrete was extremely good. Apparently it had now been adopted as general practice for concrete dams in France and Italy. In the Cornish granite, silicate of soda had been useful, but apparently in the Claerwen Dam it had not been useful in helping the grout to get into the rock. Had Mr Walton any further comments on that point ?

Mr Risbridger had referred to the question of yield, which Mr Walters had been intending to raise. The figure of 102 million gallons a day seemed to him to be wrong, or at any rate to be different from what he had expected, because the Claerwen Dam had 4,000 million gallons more storage than the three reservoirs which it supplanted, and the yield formerly envisaged had been 102 million gallons per day. However, Mr Risbridger had made it clear that it was not easy to be precise about yield.

Mr Walters asked whether a curved dam had been adopted purely for aesthetic reasons and whether Mr Morgan in particular had considered the question of a buttress dam. In a Paper presented to the Institution in 1952, Mr Roberts had given two instances where a buttress dam was about 50 per cent cheaper than a mass concrete dam,[2] whereas Mr Morgan, who had presented the Paper in Mr Robert's absence, had stated that the buttress dam appeared economical only on paper and that, owing to difficulties of joinery and shuttering, it was extremely difficult to make a real

[2] C. M. Roberts, " Fundamental Economics in Hydro-Electric Design." J. Instn Civ. Engrs, vol. 36, p. 115 (Apr. 1951).

estimate.[3] Recently Mr Atkinson, of Manchester, and Mr Taylor,[4] in discussing the Haweswater dam, had said definitely that the buttress type of dam at Haweswater, which was 134 feet high and half as long again as the Claerwen Dam, was 12½ per cent cheaper than the equivalent mass concrete dam. Any observations which Mr Morgan could make would be very interesting, because it seemed necessary to think again about the question of buttress dams. The manufacture of concrete was being revolutionized, particularly by the introduction of vacuum concrete, and that directly helped shuttering.

It would be interesting if the figure of £2,000,000 could be broken down and more information given on the main items of expenditure. Could Mr Falkiner say something about that?

Mr Walters had been struck by the fact that there appeared to have been no loss of life on the job, which seemed to show that it had been well organized. At a meeting of the British Section of the Société des Ingénieurs Civils de France, in May, 1951, when a Paper[5] had been presented on the Génissiat Dam, the members present had stood in tribute to the memory of the fifty French workmen who had lost their lives in its construction, but at Claerwen, apparently, there had been no such loss.

Mr Alan P. Lambert said that it must be a difficult choice for a designer to decide what he would assume about uplift pressure. In the case in question, it had apparently been assumed that there might be uplift of a certain intensity over the whole area of the foundation, ranging from full hydrostatic head upstream to nothing downstream. That might be the standard assumption for a gravity dam—he did not know—but had it influenced the profile of the dam at all? If the profile of the dam had been fixed entirely by other considerations the point was not worth pursuing, but it seemed to envisage a state of affairs which it was very difficult to imagine happening in practice. If it did come near to happening in practice, he thought that something much worse might happen just as easily.

If one considered water getting past a cut-off, then, ignoring the pressure-relieving drain, which was an immense safeguard, either that water came out on the downstream side of the dam or it did not. If it did not, no flow was taking place and the uplift would be the full hydrostatic head over the whole area where the water could reach. Alternatively, if it did come out, the only condition which satisfied the design assumption was if it came out at negligible velocity; but, if it got right across the foundation, might it not come out rather faster than that? Then it would have pressure head just before it emerged, and the distribution of uplift

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[3] See footnote 2, *Discussion*, p. 133.

[4] G. E. Taylor, "The Haweswater Reservoir." J. Instn Wat. Engrs, vol. 5, No. 4, p. 355 (July 1951).

[5] P. Delattre, "Génissiat Dam and the Harnessing of the Rhone." Soc. Ing. Civ. France, British Section, 105th O.G.M., 21 May, 1951.

pressure would be slightly less than the hydrostatic head upstream and possibly a good deal more than zero downstream.

Would there be any technical objection to the lower gallery having been built as an inspection passage on the rock downstream of the cut-off trench, and assuming that it was provided with suitable drainage arrangements, could one then assume that the only area subject to uplift was the area upstream of the passage? In that event, would the profile of the dam have been reduced at all?

With regard to the concrete, the most interesting feature was the use of the 6-inch aggregate, but he had been surprised to see that the initiative in that respect came from the contractor; would Mr Falkiner say what were the advantages from the construction point of view in using that 6-inch aggregate? He could see that there was some saving in the crushing costs, and, collaterally with that, some improvement in the throughput of the crushers, but apart from that he did not see what the advantage to the contractor was, and it seemed to have brought in its train some difficulties regarding workability.

In that respect, he agreed with the remark on p. 256 of the Paper that it would probably have been better to have had more grades. Six-inch and even 8-inch aggregate had been used on the Continent and in America, and there, so far as he could check, at least four grades above the sand fraction, and sometimes two grades of sand were used. That would, he thought, give a much greater control of workability.

The logical consequence of increasing the size of aggregate was to reduce the cement content, although, no doubt it would have increased the difficulties of workability. If a 7 : 1 concrete was suitable for a 2½-inch-maximum size, possibly an 8 : 1 or even a 9 : 1 concrete using a 6-inch-maximum size would have a cement grout of the same quality, because the total superficial area and the total void volume to be filled decreased with an increase in maximum-aggregate size. If the mix in question could have been reduced from 7 : 1 to 8 : 1, it would have meant a saving of 15 per cent on the cement bill. Whilst he had no knowledge of the cost of the low-heat cement, including delivery to the site, he doubted whether it would be less than £5 per ton, and at £5 per ton the saving would have been £46,000. There would also have been the advantage that, having less cement, there would be less heat generated; and even with low-heat cement, he assumed that it was desirable to keep the heat down as much as possible.

He noticed with some relish that an attempt had been made to design the aggregate grading by the use of formulae and that that had proved unprofitable. He was allergic to formulae for the design of concrete mixes, particularly when they were based upon a false premise. They all took as their criterion the grading for maximum density in the dry state, but for concrete one wanted a grading for maximum workability in the wet state, under the conditions in which the concrete was to be used on the

job; the fact that those two gradings were not the same was clearly brought out in the Paper. The Authors had finally been reduced to doing what was the right thing to do in large concreting jobs, namely, to obtain the material which they were going to use and experiment with it on a basis of trial and error, and so arrive at the right grading for the circumstances of their particular use.

Could Mr Falkiner give more information on the vibrators for the concrete? Had pneumatic or electrical vibrators proved the more suitable? What was the size of head needed with the 6-inch aggregate and how many units were necessary at each placing point?

Colonel F. C. Temple said that he was one of the few surviving members of the engineering staff that had built the first instalment of the Birmingham waterworks. He had looked with awe and reverence at the mathematical calculations in the Paper, and had been extremely interested in the comments made on them. They brought back to his mind very vividly his first interview with James Mansergh, when he had gone to see whether he could become a pupil of his. To Mansergh's question "Why do you want to be an engineer?" he had replied "I always have." He had been reading for Mods and Greats at Oxford and would probably have sat for the Civil Service examination, but when the Boer War had come, he had gone to South Africa and on his return he decided that he wanted to take up engineering.

When Mansergh had asked him why he had not gone in for engineering before, he had replied "Because I was told that I was not good enough at mathematics." Mansergh had asked "Can you add and subtract?" "Yes." "Can you multiply and divide?" "Yes." "Can you solve a quadratic equation?" "Yes." "Can you solve a simple triangle by trigonometry?" "Yes." "Then," said Mansergh, "what more do you want? Mathematics is cheap; if you want any more than that, go out and buy it. That is what I always do."

When he had gone to the Elan Valley in 1901, there had been considerable excitement there, because someone had cast doubts on the stability of the Assouan Dam, and various people had cast doubts on the stability of the Caban Dam; but James Mansergh's answer had been to add another 10 feet on top of it. If modern methods had been known at that time, that dam would have been built at least 200 feet high and would be doing nearly all the work that the other reservoirs were doing.

Returning to the question of mathematics, one of the things which anyone who liked such work could amuse himself with was working out toe curves at the back of the dam for the spill channel, and then try to set out his profiles.

Colonel Temple was of the opinion that after setting some key profiles, intermediate curves could, and should be, set out by eye.

Another aspect was the design of pipe sizes. He had regularly used a chart called the "Alexander Diagram," his copy of which had been bought

in 1905. That chart had enabled him to forestall his mathematicians every time on the question of pipe sizes, which could be obtained from the diagram with more speed and sufficient accuracy. For example, mathematics might dictate a 10-inch-diameter pipe, but no one made them ordinarily, so a 12-inch-diameter pipe would be used.

When Birmingham had asked Mansergh to provide a water supply, he had produced a scheme which he had drafted when he had been an assistant engineer building the Cambrian Railway, and it was almost impossible to make him deviate from what he had then planned. At that time he had envisaged a dam at Dol-y-mynach and two more up the Claerwen. All the engineering staff in the valley knew that the Dol-y-mynach Dam was in the wrong place. It had appalling foundations—no one bothered in those days with trial pits or with putting down bores; one went down hopefully and went on until some decent rock was encountered.

In those days it was regarded as important not to have a straight joint through the dam. As the blocks of concrete were built up, the shutters were arranged so that there would never be a straight joint. Today it was the practice to build with straight joints and to give the structure somewhere to move. Whatever structure was built, from a small house upwards, it was much better to give it somewhere to move, because it was going to move in any event, and if one gave it somewhere to move easily it would accept that and no trouble would arise. For instance, a flat roof could be put on a building without any joints on the walls, leaving it free to move. It would move with changes of temperature, but not always in one direction. Colonel Temple recalled one instance when it did move always in one direction, where on the inner side there was just a girder; in hot weather the roof expanded; when rain came it shrank, and having more adhesion on the three walls, it moved on the girder and the rain came through the crack. That had been cured by using the same kind of copper strip as was used on the Claerwen Dam.

Mr W. J. H. Rennie said he believed that the Claerwen Dam was the highest dam in England, and it was one of the few dams built in the past few years which had a masonry facing. The Birmingham Corporation were fortunate in being able to afford such a luxury, but it was architecturally in keeping with the other dams in the Elan Valley and a very welcome addition.

He was especially interested in the Claerwen Dam because his firm was one of the contractors who had been invited to submit a tender. At the time of tendering, in April 1946, he had spent some days visiting the site of the dam and exploring the countryside, looking for potential quarries for stone and sand. He had also been fortunate enough to visit the works in 1948, during the period of construction.

He thought it was good to combine in one Paper a description of the

design and construction with an account of the contractor's construction plant and methods. His only criticism was of the low ratio of 6 : 1 in the number of pages allotted to the Engineer's and the contractor's contributions. The construction work was so important that if the contractor's section could be enlarged it would further enhance the value of what was already a very valuable Paper. It had to be remembered that the contractor had the onus of spending both his own and the client's money, and it was very essential that he should do so wisely and well. It would be interesting to hear more about the contractor's problems and experiences, and some details of costs would be very welcome.

Mr Rennie mentioned some of the methods which his own firm had intended to adopt if their tender had been accepted, and showed a slide illustrating the general construction lay-out of one of the tender drawings. The approach was different in some of its aspects and also fundamentally. Instead of the use of cableways (which they had considered and abandoned), they would have provided derrick cranes elevated on gabbards 40 to about 100 feet high. It was all standard equipment and most of the plant had been part of their stock. There were four standard 1-cubic-yard mixers, each mixer commanded by two derrick cranes. The output figure of concrete mentioned in the Paper was 2,200 cubic yards per week and that could easily have been dealt with in the way shown, whilst the derrick cranes would also have coped with the excavation and masonry. The transport of materials was to have been done by Decauville wagons and track. From the quarry it had been proposed to bring stone by rail to the crushers, which were sited almost in the same position as shown in the Paper, and from the gauging station the dry-gauged materials would have gone directly to the four mixers. Upstream were the excavation spoil-heaps. They had proposed to divert the river twice; once on the right bank, and, after the left bank tunnel had been built, flumes would have been provided for diverting the water through it. The cableway had to be manufactured and its erection had been a lengthy business; by the method which his firm would have used they would have been able to start concreting about a year ahead of the time when it had actually been begun.

Would the Authors explain more fully how the 1 : 7 mix had been determined? Had the 8-cubic-foot gauge-box referred to on p. 266 been used to arrive at the 357-lb. cement weight in Table 4, and if so what was the shape of the gauge-box. He had difficulty in agreeing the cement figures for the various mixes. Would it not have been more satisfactory if the engineers had specified the various concrete mixes in pounds of cement per cubic yard of finished concrete?

Mr J. K. Hunter, referring to the possible use of a buttress dam, said that a solid concrete gravity dam was inherently extravagant in the sense that it did not take full advantage of the materials of which it was constructed; because of that, increasing attention had been given during

the past 25 years to special types of dam which secured greater economy of material.

Of the many variants of the gravity dam, the massive-buttress type possessed certain positive advantages. It was a pure gravity dam, since it owed its stability entirely to its dead weight and avoided the use of all steel reinforcement. Depending on the overall height and the details of the design, the reduction in the volume of concrete used might be as much as 35 per cent as compared with a solid dam, although a part of that economy might be offset by the use of higher-strength concrete.

Among the advantages which might be claimed for a massive buttress dam were the following :—

(1) Construction problems arising from the heat of hydration were simplified, since the exterior cooling surfaces were relatively greater than in an equivalent solid dam.

(2) The division of the dam into a series of independent buttresses provided the opportunity for dealing in a rational manner with shrinkage both along the axis of the dam and in a transverse direction.

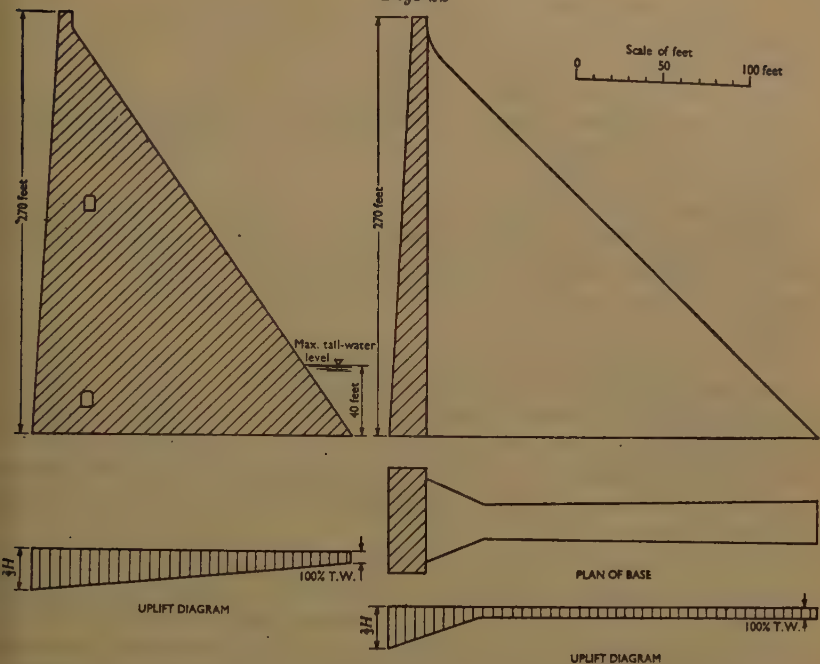
(3) The division of the dam into separate elements, each of which was freely drained on three of the four sides, limited the internal pore-pressure which could be built up both in the concrete and in the foundation rock, and therefore it was possible to adopt much less onerous uplift assumptions for design purposes ; moreover, the uplift pressures were more determinate than in the case of a solid dam and there was less margin for error.

(4) Provided the downstream face of the dam was not closed in, then the water-bearing elements and the faces of the buttresses remained accessible, and subsequent grouting of the foundation rock, if found to be desirable, could be carried out more effectively than with a solid dam.

The only disadvantage of a buttress dam as compared with a solid dam was that it was more vulnerable to earthquake shocks in a direction parallel to the longitudinal axis.

Figs 22 provided a comparison between a conventional solid concrete dam and a massive-buttress dam of simplified type. It would be observed that the design greatly simplified the shuttering problem as compared with the more complicated bull-head buttress, which involved the use of overhanging formwork. The designs had been prepared for a dam in India having a height of 270 feet. The solid dam required 804,000 cubic yards of concrete compared with 585,000 cubic yards for the buttress dam. Thus the buttress dam showed a saving of 27·3 per cent. A study of the cost of the two alternatives indicated that, after making allowance

Figs 22



	Solid gravity dam	Massive-buttress-type dam
Total volume of concrete	804,000 cu. yds (100)	585,000 cu. yds (72·7)
Cement	170,000 tons (100)	134,000 tons (79)
Relative cost	100	80·2

COMPARISON BETWEEN SOLID CONCRETE DAM OF NORMAL DESIGN AND SIMPLE FORM OF MASSIVE-BUTTRESS DAM

for the increased strength of the concrete and extra shuttering, the overall saving represented by the buttress dam would be about 20 per cent.

During the past 20 years a number of massive-buttress dams had been built in widely separated parts of the world including the United States, Switzerland, Tasmania, and the United Kingdom. The heights of those actually constructed had ranged up to 276 feet, whilst designs had been prepared for dams up to 400 feet high.

Mr Hunter also referred to the provision for uplift which had been made in the design of the Claerwen Dam. The design assumption appeared

to be 100 per cent of the hydrostatic head at the upstream face reducing linearly to zero at the downstream face, the resulting pressure being applied over the whole horizontal section.

A system of drains had been provided behind the upstream face as well as beneath the heel of the dam, but it would appear that no credit had been taken for the reduction in uplift pressure which such a drainage system might be expected to produce.

Engineers differed in their opinion of the desirability and effectiveness of a drainage system, and if there was any doubt about either the degree or the permanence of the relief afforded there was much to be said for dispensing with the drains altogether and adopting a somewhat more generous profile.

The cut-off trench beneath the heel of the dam was stated to range in depth from 15 feet to 6 feet. The use of a cut-off trench for a gravity dam on sound rock would appear to be a survival from the days before the introduction of the cement grouting of foundations. Such a trench was relatively expensive and it was often impossible to excavate to accurate lines owing to the way in which the rock broke to sloping beds and joints.

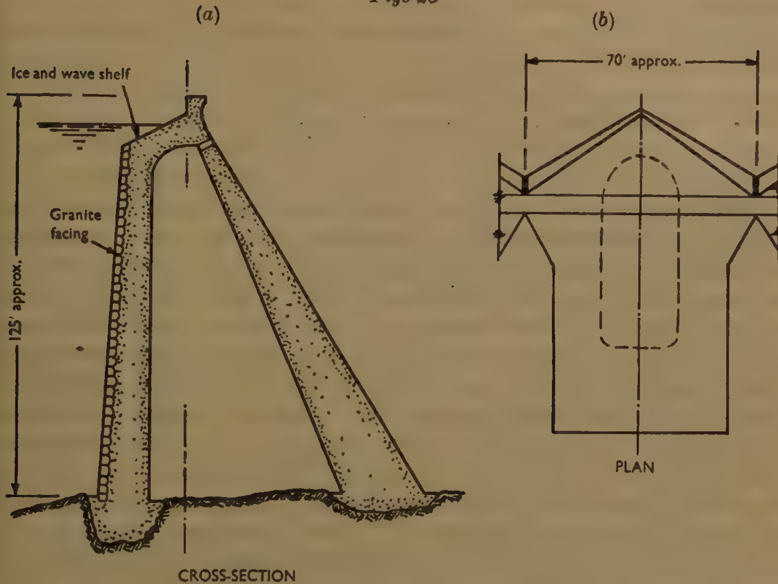
The original purpose of a cut-off trench was to replace jointed or pervious rock by more-watertight material, but provided the rock was reasonably sound and had adequate crushing strength, Mr Hunter was inclined to the opinion that the provision of an impervious barrier beneath the heel of a dam could be achieved more effectively by careful grouting. He suggested that in the majority of cases the money spent on a conventional cut-off trench could be spent better in other ways.

Mr P. O. Wolf observed that the stress distribution in a dam was the result of a more or less skillful balance of both external and internal water pressures and of the weight of the structure, and he supported previous speakers who had advocated the "open" type of dam. Its advantages were twofold: (1) for a given external load the "open" type could be designed so that most parts of the dam were fully stressed; and (2) further economies resulted from the reduction in loading owing to relief of uplift pressures on the base and in the body of the dam. He showed slides of the Haweswater Dam,[6] which was designed as a series of vertical cantilevers supporting an approximately horizontal load. Each "block" resembled, in horizontal section, a rolled steel joist—a shape which engineers recognized as very efficient. Two other slides showed details of the Inferno Dam[7] in Italy, which was similar to the Claerwen Dam in its outline (*Fig. 23* (*a*)), but as seen in the horizontal cross-section of a "block" (*Fig. 23* (*b*)) saved a weight of concrete roughly equivalent to

[6] See reference 4, p. 293.

[7] C. Marcello, "*Moderner Talsperrenbau in Italien*" ("Modern Dams in Italy"). *Schweiz. Bauztg*, Band 68, No. 33, p. 446; No 34, p. 455; No. 35, p. 476 (Aug.–Sept. 1950).

Figs 23



DETAILS OF THE INFERNO DAM, ITALY

the uplift pressure under a solid dam. In that dam it was the upstream face which was lined with masonry since, apparently, that was an economical means of protection in a country where masons were numerous.

A slide of the Loch Sloy Dam [8] presented an example of the massive-buttress type whose advantages had been described by Mr Hunter. Whilst a structure like the Haweswater Dam had continuous faces and therein resembled a massive gravity dam, the Loch Sloy Dam had great aesthetic appeal, and that was another matter of importance.

Mr Morgan had stated on a previous occasion that the apparent economy of an "open" type of dam was often not obtainable in practice because of construction difficulties owing to the extra shuttering involved, and shortages of labour and materials. It appeared to Mr Wolf that under any conditions a minimum economic gap between buttresses, etc., could be specified, and that was usually quite small. In Great Britain, where a cubic yard of concrete was more costly than a square yard of repetition shuttering, even the minimum practicable width of gap, or hollow, which might be about 5 to 6 feet, was clearly economical. In practice much larger gaps (for example, 39 feet at Loch Sloy) were common, and there the ratio of concrete saved to extra shuttering was large and highly

[8] James Stevenson, "The Construction of Loch Sloy Dam." Proc. Instn Civ. Engrs, Pt III, vol. 1, p. 169 (Aug. 1952). Mr Wolf's slide was similar to *Fig. 1* of this Paper.

economical. The extra shuttering did increase construction difficulties and unit costs of concrete, but in dams of appreciable size that was more than balanced by the saving in concrete. He showed a slide of an exceedingly simple design of dam, put forward by Mr James Williamson, M.I.C.E., in 1936,[9] which might still be considered a useful starting point for a first design today.

He regretted a remark in Mr Morgan's introduction which implied that the art should override the science in engineering, and should not be baffled by it. Mr Wolf felt that each aspect had to help the other, and whilst scientifically exact knowledge continually encroached on what in the past had been fields for intuitive decisions, the art of engineering was an essential foundation as well as the extension of the science. As an illustration he referred to the Appendix to the Paper. It contained a method of derivation of a profile for a dam whose maximum compressive stress on any horizontal plane did not exceed a given value. He was not sure if the method there suggested was preferable, in practice, to the Wegmann analysis—perhaps the views of other members would be expressed later—but he was perturbed to see a highly precise analysis applied to design assumptions so patently lacking in precision. In water-retaining structures the concrete was usually capable of taking some tension. In that case, the stress diagram in *Fig. 21* should be replaced by a straight line connecting a small tensile stress on the upstream face with a compressive stress on the downstream face which was much smaller than the limiting value q. If, on the other hand, the concrete was incapable of sustaining tension, then a loading diagram as shown in *Fig. 21* would have to be assumed to be accompanied by cracking at the upstream face. The water in the reservoir would enter the crack at full hydrostatic pressure and the uplift diagram would no longer follow the triangular pressure-distribution postulated. In fact, stress conditions as shown in *Fig. 21* would not persist. Either the maximum compressive stress on a horizontal plane, at the downstream face, would be much less than q, or much more. In practice, dams were constructed with a rich facing of less permeable concrete and drainage wells behind, and numerical analysis of practical cases confirmed the view that the uplift diagram was far from triangular and that the substitution of $(s—l)$ for s in formula (7) was unjustified.

Connected with that question were the lines of resultants shown in *Fig. 3*. Mr Wolf was surprised to see that the dam was designed for compression along the downstream face under all conditions, but for some tension on the water face when the reservoir was full and some uplift pressures developed. He considered that small tensile stresses in a gravity dam might be tolerated at the toe, but not on the water face. By a reduction

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[9] James Williamson, "Design and Waterproofing of Shrinkage, Contraction and Expansion Joints in Concrete Dams." Trans 2nd Congress on Large Dams, 1936, vol. III, p. 139. See *Fig. 9*, p. 146: "Multiple buttress type of dam to facilitate shrinkage and relieve uplift pressure."

in the amount of concrete and by making the upstream face steeper so as to allow the line of resultants (reservoir empty) to follow the limit of the middle third, a more economical and/or safer structure would have resulted.

With regard to *Fig.* 8 (*b*), he suggested that the stability of the structure could also have been improved by reducing uplift in three ways: by closer spacings of drains and pressure-relief pipes; by an increase in the size of the lower gallery, which would have permitted the use of drilling equipment, after all grouting had been completed, and possibly, in future, to clear the pressure-relief pipes and extend them into the foundation rock; and by lowering the lower gallery just above the water level in the stilling pool in order to permit escape of seepage water at lower heads.

Finally he asked whether a calibration curve could be added to the interesting profile of the measuring weir shown in *Fig. 15* (*c*), and expressed his appreciation that so many subjects of great interest had been dealt with in the Paper.

Mr Morgan, in reply, said that Mr Risbridger had referred to the discharge valves and had asked whether the Authors would do exactly the same thing again. The answer was probably that they would not. Mr Morgan would like to see a small branch and valve associated with each main valve for purposes of normal draw-off. There was a form of valve which could be operated indefinitely at part opening and that was the spear-and-orifice type used for a Pelton-wheel turbine. The needle valve was, after all, an enlarged version of such a device except that the streamlining could not be so perfect. An ideal arrangement would be to parallel the large valve with another of the turbine type for taking off the smaller discharges, and it would be possible to operate the latter over a wide range of apertures with virtually no cavitation.

On the question of buttress dams, Mr Morgan could not remember exactly what he had said on the occasion to which Mr Walters had referred, but he did not think Mr Walters had recalled the conversation quite accurately. Mr Morgan still did not believe that the buttress dam would be as much as 50 per cent cheaper, but it could be substantially less costly in certain circumstances. The essence of the question was how much the shuttering cost. It was all very well to arrange for void spaces in dams, but shuttering was necessary and had to be paid for; its cost varied in different parts of the world. If he were told where the site was he would be able to say whether a buttress dam was likely to pay. In Great Britain, in recent years, no form of shuttering had been easy to obtain. The buttress dam required a great deal more shuttering, and it was no use saying that there would be a saving of 35 per cent of concrete, because that left the shuttering completely out of the question. On the other hand there was no doubt that the buttress dam was very much a thing of the future. It had been considered in more than one case recently. It was a very good type of dam and there was no doubt that it would gain ground. He

thought that a $12\frac{1}{2}$-per-cent saving in cost might be nearer the truth generally, but it was in fact something about which it was not possible to generalize.

Mr Lambert had referred to uplift pressure. The Claerwen dam had been built on a comparatively pervious foundation. Mr Morgan had now designed a number of dams of about the same size, and two of them, the Claerwen dam being one, had been on a foundation of that kind. That was the chief reason for the profile. It was a question of to what extent it was possible to rely on relief from the drainage system. It was very good while it was all effective but it could get choked. The ultimate question was one of probability. That was why the assumption had been made of full uplift at the upstream face. He did not see the same picture as Mr Lambert, of a sheet of water shooting under the dam and emerging at the toe. It did not do that; it came out somewhere else, usually at a point further downstream. He did not like the suggestion of putting a drainage gallery right down in the cut-off, if that were in fact what was suggested. It could be an inspection gallery only after it had been pumped out, because it would be full of water the whole time. He thought that the inspection gallery was very well where it was.

He had considerable relish in answering Colonel Temple. He had made it clear that mathematics for the sake of mathematics was not something of which he approved; like dynamite, it was very useful, but not if it took charge of one; but he thought that Colonel Temple in all his travels, both before and after the Boer War, would not have liked the ship's officers to fix their position at sea by eye. Mr Morgan disliked the old saying about buying mathematics. One could, of course, buy mathematics, but if unable to follow the analysis the purchaser would not know what he was buying.

Mr Morgan did not agree with Mr Wolf. So far as the assumption of no tension was concerned, *Fig. 21* was intended simply to illustrate the principle and not to be a true-to-scale diagram of the forces acting on the dam. It was quite common to assume that there was no tension, as in reinforced concrete design, but that did not mean that one assumed that the whole dam would crack from end to end along that line. It was elementary that if one did put in the tension one reduced the maximum compressive strength. Members had all learned that a long while ago and it was hardly necessary to point it out.

The assumption of no tension in the concrete was fundamental, for instance, to the design of reinforced concrete but that did not carry with it the implication that large cracks would open on the tension side.

Messrs Scott and Walton, in reply, referred to Mr Walters's mention of silicate of soda. That had certainly been used with great effect in disintegrated granite to lubricate fissures which were of material size, but in the case of Claerwen the fissures were very minute and the silicate of soda had been completely without effect. In a large number of cases the

grains of cement had been too large to enter the fissures at all. The discharge of the pressure-relief drain indicated extreme tightness, and the Contractors had got very little leakage into the foundation when they had had the full area open. The curve of the dam, as indicated in the Paper, had been adopted purely for aesthetic reasons.

The split-up of the £2,000,000 was as followed :—

	£
Main Dam	
Excavation	67,800
Pressure grouting	15,000
Mass concrete, including forming tunnels, etc. .	1,571,000
Brickwork to upstream face,	
Extra over concrete	23,800
Masonry on faces,	
Extra over concrete	182,900
Draw-off pipes and valves	42,500
	1,903,000
Other Works (including masonry facing)	
Spillway channels	67,800
Viaduct	66,700
Bridges	8,600
Roads and approaches	19,000
	£2,065,100

The Authors would give the warning that those costs might be somewhat misleading. It was like the balancing up of a buttress and a gravity dam, in that if one had not put masonry on the facing it would have been necessary to do something else, and how much would that have cost?

In reply to Mr Lambert's point about the cement content, it had been found on the volumetric tests between a 6-inch and a 2½-inch aggregate that 15 lb. of cement per cubic yard was theoretically saved to give the same 7 : 1 mix, due to the grading.

The assumption that an increase in the maximum size of the aggregate would allow for a reduction in the 7 : 1 volumetric proportion without reduction in strength of the resulting concrete was incorrect.

The increase in size of aggregate made a reduction in the quantity of cement per cubic yard of concrete without altering the 7 : 1 proportion, or the equivalent strength. That was because of the variation in the weight of materials in one cubic foot of small aggregate mixed dry as compared with one cubic foot of large aggregate mixed dry. In the case of the Claerwen aggregate that reduction in cement had been about 5 per cent, of which advantage had been taken. The amount of reduction depended on the particle shape of the stone and might have been quite different with a gravel aggregate.

Messrs Scott and Walton wholly agreed with what Colonel Temple had said about the difficulty of setting out downstream toe curves. For instance, even if two profiles were set up perfectly correctly, a line strung between them would still not be in the plane of the face unless it was also strung on the correct grade.

Mr Rennie had mentioned cableways versus derricks. It was of interest that in the correspondence on Mr Mansergh's Paper[1] it was mentioned,[2] in answer to one contributor, that aerial ropeways had been considered but had been ruled out owing to the conditions prevailing.

The method of determining the 1 : 7 mix had been by placing (by hand) mixed aggregate into the gauge box, which was a 2-foot cube. The materials were weighed as they were placed into the box so that it was possible to know the actual weight of aggregate corresponding to 8 cubic feet. The volumetric results were averages after numerous tests had been carried out in the manner agreed with the Contractors.

Even if concrete mixes were specified by weight of cement per cubic yard the volumetric density of the particular aggregate used would still have to be obtained.

The calibration curve for the measuring weir as obtained from the model experiments was $Q = 218{\cdot}68\ H^{1{\cdot}68}$. The term L was not apparent, since it was a variable depending on the battered sides of the channel. The length of the weir was about 77 feet.

Mr Falkiner, in reply, said that there was no real evidence as to whether 6-inch aggregates contributed to unworkability. The problem at Claerwen had been caused more by the rough-fractured surfaces of the particles, and it might well be that the smaller the aggregates the greater the difficulty, other factors being equal.

The advantages of the large-sized aggregates were threefold, namely: (*a*) reduced cost of production (as mentioned by Mr Lambert); (*b*) reduced cement and sand content in concrete; and (*c*) increased weight of concrete.

The great disadvantage was the tendency for particle segregation when tipping both dry aggregates and wet concrete. When that could be overcome, it seemed uneconomic to spend money breaking stone and more money on cement to stick it together again.

Mr Lambert had also mentioned concrete-vibrators. Three types had been used:

C.P.417.	compressed air	4¼ inches dia., weight 55 lb.
I.R.3V.	compressed air	4¾ inches dia., weight 78 lb.
Allam.	Electric type E.F.	3½ inches dia., weight 18 lb.

The first had been satisfactory but had had to be abandoned owing to spare-part shortages. The second had been too heavy, requiring two men to handle it. Most of the work had been done with the third type, it being found desirable to use 400-volt 3-phase motors and not single-phase types.

Mr Rennie's description of a competitive method of construction was of great interest to Mr Falkiner. However, if the Contractors had had to make the decision again they would definitely select cableways in preference to derricks, with one central mixing plant, if only for the saving in labour

[1] See footnote 1, p. 249.

[2] *Loc. cit.*, p. 86.

cost, estimated at £80,000. That sum was more than enough to purchase and erect the cableways and other special plant. The work had been planned and carried out at a time when labour shortage was the greatest difficulty foreseen and experienced, and for that reason considerable delay would have been experienced in the initial stages whatever scheme had been adopted.

The erection of cableways had taken 5 months, but there was a compensating advantage in that the work could be done without interfering with excavation. It would barely be possible to erect gabbards and derricks at the edge of the excavated area until the heavy blasting necessary there had been completed. Another disadvantage of the derrick scheme was the necessity for those machines to luff with their load, which would greatly reduce output. Finally, it seemed from the scheme outlined by Mr Rennie that the derricks could not command the whole area of dam at their full-load radius, nor could they (on gabbards 100 feet high) reach high enough to complete the structure, which was 210 feet above stream-bed level. With those limitations it would have been impossible to reach the high peak-outputs of 90/100 cubic yards per hour attained by the cableways, and without those the average would have been greatly diminished owing to weather conditions and the continuing reduction of scope as the thickness of the dam diminished towards the top.

Referring to difficulties and how they had been overcome, the most formidable had been the collection of suitable men. The years 1946 to 1948 had been notably a period of very full employment, and the competition for good men had been at its height. The problem had been solved by recruiting Poles from the Resettlement Corps and Irish in Ireland, most of the latter being British ex-servicemen. Men had been encouraged, by good camp conditions, to settle down and the change-over of men had been remarkably low. Many of the skilled men had been of local origin, specially trained for their job, and most of the drivers had served for the whole period.

Other difficulties, such as the problem of producing good concrete with the aggregates available and the shortage of masons and masonry had been dealt with in the Paper.

Correspondence on the foregoing Paper is closed and no contribution, other than those already received at the Institution, can now be accepted. —Sec. I.C.E.

ELECTION OF MEMBERS AND ASSOCIATE MEMBERS

The Council at their meetings on the 20th January and the 17th February, 1953, in accordance with Bye-law 14, declared that the undermentioned had been duly elected as :—

MEMBERS

Abroad

Robert Knight Innes
James William Restrick, B.Sc. (*National*).
Baron Willem Francis Van Asbeck

ASSOCIATE MEMBERS

Home

William Henry Adams, B.Sc. (Eng.) (*Lond.*).
Ronald James Amblin, Stud.I.C.E.
Harold Douglas Bagley, Stud. I.C.E.
Frederick Vincent Scott Kuenanz Bayliss, B.A. (*Oxon.*), Stud.I.C.E.
Francis Roy Bishop.
Charles William Bowman, Stud. I.C.E.
Paul Frederick Boyce, Stud.I.C.E.
John William Bradshaw.
Leslie Bullock.
Peter Frederick Bullock, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
John Munro Campbell, B.Sc. (*Glas.*), Stud.I.C.E.
Alfred Carlo Cassell, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Reginald Walter Cluett.
Henry Vince Colley, Stud.I.C.E.
Alan Reginald Cox, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Dennis Cox, B.Sc. (*Wales*).
Robert Campbell Craig.
Anthony Gale Durrant, B.Sc. (*Bristol*).
Kenneth George Waddington Field.
John Francis Fleming, B.Sc. (Eng.) (*Lond.*).
David Christopher Brontë Gatenby, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
Roy Gifford, B.Eng. (*Liverpool*).
Peter Malcolm Grant, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Donald William Green, B.A. (*Cantab.*), Stud.I.C.E.
Norman David Hardie, B.E. (*New Zealand*).
Peter Vernon Harvey, B.Sc. (*Durham*).
Frank Ernest Jarvis, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
John Johnson, B.Sc. (*Durham*).
Kenneth Herbert King, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Edward Stewart Edmund Lee, Grad. I.C.E.
Alan Gordon Lewton, B.Sc. (*Birmingham*), Stud.I.C.E.
Cyril Llanwarne, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
Dugald James McKillop.
Maurice Francis Maggs, B.Sc. (*Bristol*).
Alexander Ritchie Murray, B.Sc. (*St Andrews*), Stud.I.C.E.
John Antony Neill, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Peter Bramwell Nissen, B.E. (*New Zealand*).
Kenneth Malcolm Odell, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Vernon Thomas Pannell.
Frederick John Parker, M.Eng. (*Liverpool*), Stud.I.C.E.
John Warrington Parnham.
Peter Kendrick Parrott, B.Sc. (*Bristol*).
Roy Pilcher, Stud.I.C.E.
Henry Howard Prichard, B.Sc. (*Wales*).
Tom Ridley, B.Sc. (Eng.) (*Lond.*), Stud.I.C.E.
Charles Walter Roberts, B.A., B.A.I. (*Dublin*).
Herbert Henry Nelson Robinson, B.Sc. (*Belfast*), Stud.I.C.E.
Frank Arnold Rowbotham, B.Sc. Tech. (*Manchester*).

NAGALINGHAM SARAVANAPAVANANTHAN, B.Sc. (Eng.) (*Lond.*), Grad.I.C.E.
JAMES SMART.
GERALD ALBERT JAMES STRONG, Stud. I.C.E.
PERCY WILLIAM TANNER.
GEORGE ALFRED EDWARD THOMAS.
PETER ALEXANDER TOOLE.
DONALD JAMES TREWEEK, M.A. (*Cantab.*), Stud.I.C.E.
JOHN DEACON TROTMAN, B.Sc. (*Bristol*), Grad.I.C.E.
JACK ROLAND TWYDLE.

ASSOCIATE

Home

ALFRED ROBERT LEE, Ph.D.

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

John Boyd Brodie (E. 1895, T. 1913).
James Cross (former Member of Council) (E. 1939, T. 1945).
Charles Henry Gale (E. 1889, T. 1913).
Frederick Harry Greenhough, D.S.O. (E. 1897, T. 1911).
Horace John Elliott Hone (E. 1910, T. 1936).
William Rowand McKim (E. 1914, T. 1938).
William Dick McLaren (E. 1910, T. 1925).
Nathaniel Martin, B.Sc. (E. 1910, T. 1943).
Stuart Kilsby Mobbs (E. 1912, T. 1935).
Brigadier-General Magnus Mowat, C.B.E., T.D., F.R.S.E., (E. 1901, T. 1909).
Charles Butterworth Newton (E. 1894, T. 1899).
Professor Norman Augustus Victor Piercy, D.Sc. (E. 1930, T. 1936).
Albert Hedley Quick (E. 1904, T. 1928).
Leslie St. Clare Rundlett (E. 1926, T. 1931).
Sir Leopold Halliday Savile, K.C.B. (Past-President) (E. 1895, T. 1914).
Edward John Silcock (E. 1887, T. 1901).
Andrew Wilson (E. 1927).

Associate Members

Silvanus Harold Arthur Abbott, B.Sc. (E. 1946).
Cecil Edward Barnes, D.S.O., M.C., B.E. (E. 1948).
George Edward Bell, B.Sc. (E. 1915).
Hugh Stowell Cregeen (E. 1904).
Leonard William Elliott (E. 1946).
William Geden Johnson (E. 1915).
Wilfrid Drake Lancaster (E. 1911).
John Haggie Patterson (E. 1908).
Walter William Robert Roche (E. 1945).
The Rev. Roland Harry Streatfeild (E. 1907).
Henry Ambler Whitaker, M.Eng. (E. 1929).

Student

Derek James MacKenzie (A. 1948).

Paper No. 5879

"Quality Control of Concrete—Its Rational Basis and Economic Aspects"

by

Niels Munk Plum, M.Dan.Inst.C.E.

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

The most economical concrete construction is obtained when the initial costs, plus the costs for maintenance capitalized over the whole lifetime of the structure, are minimum. The cost of the concrete materials is nearly proportional to the average strength, and the Paper shows that if the homogeneity of the concrete is improved, the average strength may be lowered without impairing the actual reliability of the structure. The saving in the cost of materials obtained by lowering the average strength is counteracted by the increase in costs for equipment, labour, and control, which is necessary to improve the homogeneity, and a minimum of the initial costs must therefore exist.

After a brief presentation of the causes of variations in concrete quality and methods for calculation of the standard deviation within a batch and from batch to batch, the first half of the Paper is mainly concerned with a theoretical determination of this minimum, but some thought is also devoted to the question of minimizing the total costs (including the capitalized maintenance costs).

In the second part of the Paper, the actual concreting process is analysed through all stages, and a number of suggestions as to the existing possibilities of improving of the homogeneity, in order to minimize the total initial costs, are offered.

INTRODUCTION

THE quality which a concrete should have in order to be used for a certain purpose must be determined from economic considerations, to the effect that the initial costs and the maintenance costs together will be as low as possible.

Today this is not secured by the various codes and standards governing the use of concrete; they are enforced to protect the consumers against unreasonably low strength, density, etc., but this does not necessarily entail a minimizing of the total costs. The explanation is that the durability of concrete depends not only upon the average value of strength or density, but also upon the homogeneity, that is, the dispersion of these quality characteristics.

† Correspondence on this Paper should be received at the Institution by the 15th September, 1953, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

There is a general tendency to improve the homogeneity of the concrete on the basis of the results obtained in laboratories and on building sites under rational control. Improvement of this aspect of quality is generally accompanied by an increase of the costs of equipment, raw materials, supervision, and testing; such a general increase of overhead costs at the site is justified—from an economic point of view—only if there are possibilities for full or more than full compensation in the form of either utilization of the improved properties of the concrete, or a reduction in the total capitalized maintenance costs of the building. Utilization of improved homogeneity of the concrete may take place either by using less cement per cubic metre—thus lowering the average quality of the concrete—or by reducing the dimensions and thus the consumption of concrete. In either case, initial costs of the construction are reduced.

Incomplete control methods and lack of statistical understanding have, in some cases, led to the contrary of that aimed at, namely, a larger increase in the initial costs than it was possible to compensate by the improvement of the quality.

The object of the Paper is to throw some light on the relations between the various qualities and costs mentioned above, so that one day it might be possible to create such new codes and standards as will form the correct basis for the economical use of concrete.

Before entering into these questions, however, it will be necessary briefly to consider certain definitions for the variation of quality.

Measures of Variability

Variations Within a Single Batch

If all the concrete in a single batch is poured in test specimens of uniform shape, and if the compressive strength, x, is determined at a given age, the results may be distributed as illustrated by the stepped histogram in *Fig. 1*. Usually the stepped curve may be approximated by the continuous Gaussian distribution, which is also shown in *Fig. 1*. The Gaussian distribution is completely defined by the two parameters: the mean and the standard deviation.

An estimate, δ_1, of the standard deviation for the batch is obtained by calculating:

$$\delta_1 = \sqrt{\frac{1}{n-1}\sum^{n}(x - \bar{x}_1)^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

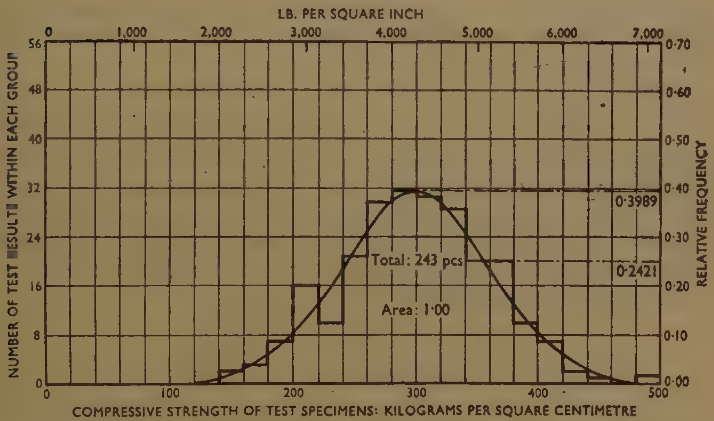
where $\bar{x}_1$ denotes the mean value of the n test results. In the case illustrated in *Fig. 1*, $n = 243$, $\bar{x}_1 = 4{,}270$ lb. per square inch (= 300 kilograms per square centimetre), and $\delta_1 = 854$ lb. per square inch (= 60 kilograms per square centimetre).

Instead of expressing the standard deviation in absolute measures,

such as lb. per square inch or kilograms per square centimetre, it may also be expressed in relation to the mean value. It is then called the "coefficient of variation" and is generally expressed as a percentage of the mean value.

The coefficient of variation will often be constant for the same process

Fig. 1



Distribution of Compressive Strengths for 243 Concrete Specimens

(The ordinate at left indicates the number of test results within the intervals at the abscissa. The ordinate at right indicates the relative frequencies corresponding to the Gaussian distribution which is also shown.)

and equipment irrespective of the strength level (of course, within reasonable limits), and therefore it will often be natural to use it as a characteristic of the variation.

How much the results within one batch will vary around the mean level will depend upon the type of concrete-mixer employed.

Total Variations Within and Between Batches

When the total variability of the concrete intermittently produced (in batches) at a site is considered, this variability may be traced back to two fundamentally different sources :—

(1) Variations within the individual batches mainly caused by imperfect mixing, segregation during discharge operations, and variation in testing conditions.

(2) Variations in average quality from batch to batch caused by imperfect control of mix proportions and by trend variations in the quality of cement and aggregates.

Since the steps to be taken to improve the homogeneity of future output of concrete will depend upon whether the main source of the total variation

is attributable to (1) or (2), it is important to separate the two components of variation and compare their relative magnitude in the statistical analysis of data from the site.

This may conveniently be done by using the statistical method known as the analysis of variance, provided the sampling procedure is arranged accordingly. A description of the analysis of variance may be found in most modern handbooks of statistics and only a brief sketch of the procedure is presented here. Test specimens sampled from the current production of concrete on a site are considered, and it is assumed that the following sampling procedure is used.

Samples of n specimens are taken at specified time intervals at a given stage of the production process, each sample of n specimens originating from an identified batch of concrete. The size and shape of the specimens, their handling, and the procedure of testing for an important quality characteristic (compressive strength, for instance) are standardized.

Suppose k batches are sampled. The value obtained from the test of the ith specimen from the jth sample (that is from the jth batch in the order of sampling) is indicated by :

$$x_{ij} \begin{pmatrix} i = 1, 2, 3 \ . \ . \ . \ n \\ j = 1, 2, 3 \ . \ . \ . \ k \end{pmatrix}$$

Within-Batch Variation for Several Batches

As a measure of the degree of homogeneity within the jth batch (in the above-mentioned sense), the estimated standard deviation, δ_j, is used, calculated in the usual way :

$$\delta_j = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(x_{ij} - \bar{x}_j)^2} \quad . \ . \ . \ . \ . \ (2)$$

where $\bar{x}_j$ denotes the mean obtained for the sample from the jth batch.

$$\bar{x}_j = \frac{1}{n}\sum_{i=1}^{n} x_{ij} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ (3)$$

The precision of an individual estimate, δ_j, is normally not very high, because the number of specimens in the sample traditionally is small ; for most sampling plans, $n = 3$.

However, from the k batches, k estimates of the respective within-batch standard deviations are obtained, and if the batches are produced under similar conditions it may be assumed that the true within-batch standard deviations are the same for every batch *[1] ; on this assumption

* The assumption of uniform within-batch standard deviation may be tested by Bartlett's test, see for instance reference 1.

[1] The references are given on p. 333.

a more reliable estimate, δ_w, of within-batch standard deviation may be obtained by pooling the k independent estimates according to the usual rule :

$$\delta_w = \sqrt{\frac{1}{k}\sum_{j=1}^{k}\delta_j^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

δ_w denotes an over-all estimate of the degree of homogeneity within individual batches, at the stage of production, where the sampling takes place.

Batch-to-Batch Variation

If within-batch variation was the only source of variability in the data, it would be possible to form another independent estimate, δ_b, of within-batch standard deviation of test specimens by calculating :

$$\delta_b = \sqrt{\frac{n\overset{k}{\Sigma}(\bar{x}_j - \bar{\bar{x}})^2}{k-1}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$(j = 1, 2, 3 \ . \ . \ . \ k)$$

where $\bar{x}_j$, as before, denotes the mean for the jth batch, and $\bar{\bar{x}}$ the overall mean for all batches.

$$\bar{\bar{x}} = \frac{1}{nk}\sum^{n}\sum^{k} x_{ij} = \frac{1}{k}\sum^{k} \bar{x}_j \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

A test on the hypothesis that within-batch variation is the sole source of variability, is performed by calculating the variance ratio, v^2 :

$$v^2 = \frac{\delta_b^2}{\delta_w^2}$$

If v^2 is significantly greater than 1, it may be taken as evidence that extra causes of variation, that is, variation in average quality from batch to batch, are present. The procedure for judging the significances of v^2 is described in handbooks of statistics, see for example reference 1.*

Assuming v^2 to be significantly greater than 1 (this will nearly always be the case), it is of interest to obtain an estimate of the standard deviation corresponding to the variation of average values from batch to batch. Such an estimate, ω, is obtained by calculating :

$$\omega = \sqrt{\frac{1}{n}(\delta_b^2 - \delta_w^2)} = \sqrt{\frac{\overset{k}{\Sigma}(\bar{x}_j - \bar{\bar{x}})^2}{k-1} - \frac{\delta_w^2}{n}} \quad . \quad . \quad (7)$$

* In English statistical literature the variance ratio, v^2, is sometimes denoted by e^{2z} or F.

The corresponding coefficient of variation is expressed as :

$$V_\omega = \frac{\omega}{x} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

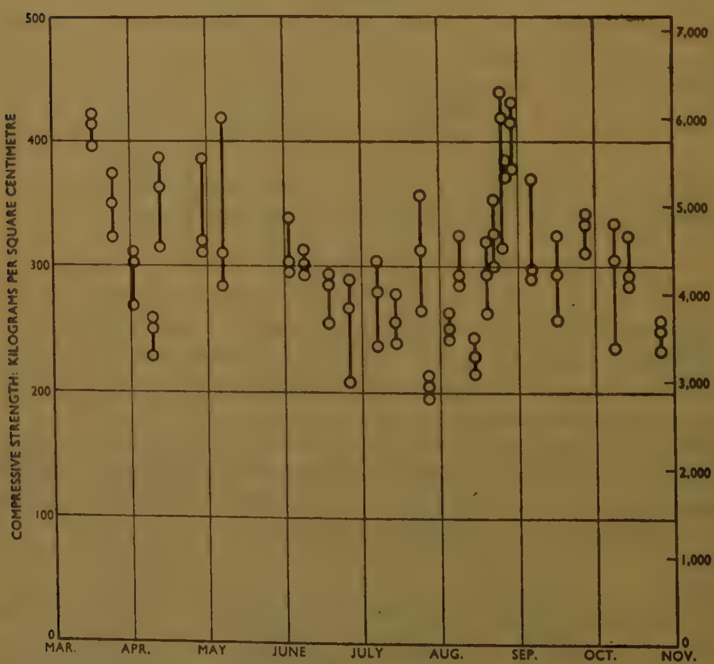
It is thus necessary to distinguish between the two variations, δ_w and ω, and to take both of them into consideration, because the total variation, which must be imagined as the variation between all the test specimens which could be cast from all the batches produced, will be a combination of these two variations.

As a measure of this total variation, the square root, s, of the sum of the two variances $\delta_w{}^2$ and ω^2 may be used :

$$s = \sqrt{\delta_w{}^2 + \omega^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

This—with a slight correction—corresponds to the standard deviation, which would have been found if, in the absence of the knowledge of the origin of the results, all of them had been dealt with in one lot.

Fig 2



Compressive Strengths of Individual Concrete Cylinders at 7 Days, from the Runways of a Danish Airfield

Variations Occurring in Practice

An impression of the variations which will occur in practice when concreting can be obtained from, for instance, *Fig. 2*, which shows results from test specimens taken at intervals of about a week during the construction of airfield runways in Denmark. Three results from each sampling are plotted vertically above each other and connected by a straight line. This provides a visual impression, partly of the variation between test specimens made at the same time and in the same place, and partly of the variation occurring in the running production from time to time.

Comparing the two kinds of variations, it will be seen—and it will also generally be the case—that the difference in quality from batch to batch will be essentially greater than within the individual batch, even if the variations inside the individual batch are not inconsiderable.

Extensive information on this question has been published in a Paper [2] on Heathrow airfield.

If the recordings in *Fig. 2* are compared with the data from Heathrow, it should be recalled that the latter indicate the variations of the *mean* figures from time to time only. An analysis of the individual results at Heathrow made by the Author shows that the variations in strength:

for test specimens within a batch, correspond to a standard deviation of about $2\frac{1}{2}$ per cent;

for test specimens made at the same time and in the same place (but from different batches), correspond to a standard deviation of about 5 per cent; and

for test specimens made at different times and in different places, correspond to a standard deviation s of about 10 per cent; it is only this last-mentioned variation which appears from the report.

As previously mentioned, the standard deviation, δ_w, inside a batch depends upon the homogenizing capacity of the mixer. The testing of Danish concrete-mixers, just completed by the Danish State Testing Laboratory and the Danish National Institute of Building Research, briefly shows that the existing types will give the coefficients of variation indicated in Table 1.

For comparison, Table 2 relates " guesses " of coefficient of variation (V_ω) from batch to batch for different methods of execution according to Stanton Walker.[4]

Table 3 indicates the coefficients of variation which are to be found for certain other building materials.

Care must be taken in interpreting information as to standard deviations obtained, since it is sometimes difficult to distinguish between δ_w, ω, and coefficients of variation. The impression is often obtained that ω standard deviations are δ_w standard deviations, because the individual values indicated are actually the means of, for instance, three values from the same batch.

TABLE 1

Coefficient of variation, $V\delta_w$ within the same batch		Cement content : 253 lb./cu. yd (150 kg./cu. m.) Slump : < 0·4 inch (< 1 cm.)	Cement content : 505 lb./cu. yd (300 kg./cu. m.) Slump : < 0·4 inch (< 1 cm.)
	4%		Positive mixer with vertical axis
	5%	Positive mixer with vertical axis Non-tilting "Kaiser" mixers	Non-tilting "Kaiser" mixers
	6%		
	7%		
	8%	Ordinary non-tilting mixers	
	9%		Ordinary non-tilting mixers

For further information see Reference 3.

TABLE 2

Coefficient of variation	Kind of operation
5 per cent	Probably attainable only in well controlled laboratory tests.
10 „	Excellent, approaches laboratory precision.
12 „	Excellent
15 „	Good
18 „	Fair
20 „	Fair minus
25 „	Bad

TABLE 3

	Large consignments	Same charge
Round mild steel ¼–½ inch diameter (7–12 mm. „)	about 10 per cent	about 5 per cent
⅝–¾ inch diameter (16–20 mm. „)	„ 6 per cent	„ 3 per cent
Danish and Swedish fir	about 20 per cent	
Building bricks	about 25–30 per cent	about 20 per cent

On the other hand, many statistics may be unnecessarily pessimistic, for instance, those from the German *Reichsautobahnen*,[5] where a coefficient of variation of 23 per cent is found from a total of 15,000 results—the standard deviation on the individual construction sites must have been essentially smaller.

Minimizing the Total Costs

In the latest editions of the reinforced-concrete specifications of most countries, provisions have been made for manufacturing several different qualities of concrete. The differences will result from the use of somewhat different methods and materials, and generally they must first and foremost be secured by different degrees of control. The question will arise, irrespective of what way is chosen, as to whether the increased expenses involved by:

(*a*) better methods of production (better equipment),
(*b*) more careful and more effective control, and
(*c*) better materials,

will finally, when they are distributed over the whole lifetime of the structure, be smaller or larger than the reductions in the annual expenses resulting from the smaller amount of maintenance work and/or a longer lifetime resulting from the improvements.

On certain types of work it is probable that the improved homogeneity of the concrete, obtained by means of better equipment and control, will be able to reduce the annual maintenance costs so much that an economical profit is involved in producing concrete of the highest quality.

On other types of work, for instance, residential buildings, it does not necessarily follow that a reduction of the maintenance costs can be obtained by improved equipment and control. In this field, improved economy will thus only result if the building authorities allow a sufficient reduction of dimensions.

The Most Economical Homogeneity

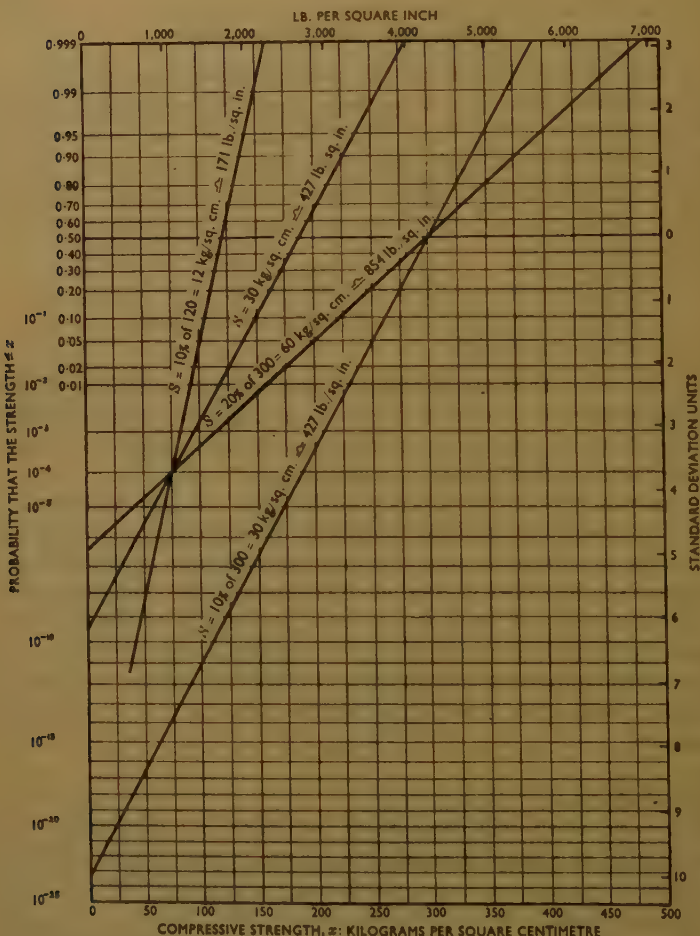
Extensive work still remains to be done, by laboratories and by the collection and adaptation of data for use in practice, before directions for minimizing the total costs, which are generally applicable in practice, can be established.

In order to give an impression of the possibilities and the economical aspects of the matter, a very simplified example, based upon the assumptions briefly summarized in the Appendix, is reviewed in the following.

Strength.—The possibilities for saving, when the homogeneity of the concrete is improved, are illustrated in *Fig. 3*, where the two straight lines on the right show the theoretical probabilities of failure for two sets of

concrete compressive-strength test-specimens having the same average compressive strength (4,270 lb. per square inch = 300 kilograms per square centimetre), but different coefficients of variation, 10 and 20 per cent, corresponding to the standard deviations 427 and 854 lb. per square

Fig 3



inch (30 and 60 kilograms per square centimetre). The two lines intersect at the abscissa 4,270 lb. per square inch and the ordinate 0·50.

If a " factor of safety " of 4 is used when establishing the permissible stress, this stress will for both sets of specimens be 1,070 lb. per square inch (75 kilograms per square centimetre).

When considering the ordinates corresponding to 1,070 lb. per square inch, it will, however, be seen that the probabilities of failure of the two concretes at this permissible stress level differ very much, since the concrete with a coefficient of variation of 20 per cent has a probability of failure of 10^{-4} at 1,070 lb. per square inch, whereas the concrete with a coefficient of variation of only 10 per cent has a probability of failure of about $10^{-13 \cdot 5}$ at the same level.

It sounds like a paradox to say that these two concretes have the same "factor of safety," because the concrete having the small coefficient of variation is quite obviously far more reliable than the concrete with the large coefficient of variation.

Since most standard specifications, so far as can be judged, assume that the majority of all concrete has a coefficient of variation of about 20 per cent and consequently a probability of failure of 10^{-4}, or a "reliability" of $1 - 10^{-4}$, it will be natural to let the reliability of $1 - 10^{-4}$ be the starting point of the following considerations.

Assuming this reliability to be 1,070 lb. per square inch, the concrete with a standard deviation of 427 lb. per square inch has, as will appear from *Fig. 3*, an average compressive strength of 2,680 lb. per square inch (188 kilograms per square centimetre), and a concrete with a coefficient of variation of 10 per cent will have an average strength of only 1,710 lb. per square inch (120 kilograms per square centimetre). It is remarkable that such a decrease of the average value (to 1,710 lb. per square inch) corresponds to a reduction of the so-called factor of safety from 4·0 to $\frac{1710}{1070} = 1 \cdot 6$, without reducing the reliability for the stresses which actually occur.

It will appear from *Fig. 3* that the use of a coefficient of variation of 10 per cent at this lower strength level means that the absolute standard deviation will drop from 427 to 171 lb. per square inch (from 30 to 12 kilograms per square centimetre).

From existing investigations concerning the background of the standard deviation of concrete it cannot be decided, whether the coefficient of variation or the standard deviation remains constant, when the strength level is changed. To be on the safe side it is therefore perhaps necessary to assume the standard deviation constant (427 lb. per square inch). *Fig. 3* shows that, in this case, the not insignificant reduction of the average strength to 2,680 lb. per square inch is obtained.

Cement content.—The above-mentioned reductions of the average strength from 4,270 lb. per square inch to between 1,710 and 2,680 lb. per square inch will allow reductions of the average cement content of practically the same size, namely, from 505 to between 203 and 317 lb. per cubic yard (300 to 120–188 kilograms per cubic metre).

These latter figures are smaller than the minimum requirements of most

standard specifications in respect of concrete of a high quality, and consequently are not acceptable.

On the basis of existing experience, it is hardly possible quantitatively to decide what limit the durability imposes on the saving in cement found by the strength considerations. This is partly because the relation between durability and cement content is not fully established, and partly because the possibilities for replacing part of the cement by filler have not been sufficiently explored. However, in principle, it is clear that a considerable saving in cement can be made.

With regard to the economical consequences of a reduced standard deviation and the consequent possibility of a smaller average strength, the fact is that the reduction of the standard deviation will generally cost money, whereas by the reduction of the average strength as mentioned above, a saving in the cement content of about 0·12 lb. per cubic yard per 1 lb. per square inch (about 1 kilogram per cubic metre per kilogram per square centimetre) can be obtained.

It has not yet been possible to find, in published Papers, thoroughly reliable figures for extra expenses involved by a reduction of the coefficient of variation by, for instance, 5 per cent. This is, no doubt, amongst other things, one consequence of the facts that: (*a*) it is cheaper to reduce the coefficient of variation, say from 30 to 25 per cent than from 10 to 5 per cent; and (*b*) within each individual interval it is possible to choose between several different kinds of procedures.

A conservative estimate of the extra expenses for reduction of the standard deviation from, for instance, 20 to 15 per cent by means of better equipment and control, would be between 1*s*. 8*d*. and 3*s*. 4*d*. per cubic yard (2 to 4 Danish Kroner per cubic metre) or from 4*d*. to 8*d*. per cubic yard (0·40 to 0·80 Danish Kroner per cubic metre) for each per cent.

The possible saving, by reduction of the average strength from 4,270 lb. per square inch to 1,170 or 2,680 lb. per square inch, will be 304 to 189 lb. of cement per cubic yard (180–112 kilogram per cubic metre) or about 15*s*. 2*d*. to 9*s*. 6*d*. per cubic yard (about 18·00 to 11·20 Danish Kroner per cubic metre) for a 10 per cent reduction of the standard deviation or 1*s*. 6*d*. to 11*d*. per cubic yard (1·80 to 1·12 Danish Kroner per cubic metre) for each per cent.

If the reduction of the standard deviation is continued below 15 per cent, more and more expensive methods must gradually be employed, and since the possible saving in cement will be more and more doubtful, a limit will finally be reached, beyond which it will generally not pay to homogenize.

Within the interval, where the concrete standard deviation today is generally lying, the result is, however, as will be seen, that, for each per cent of reduction, a saving of between 1*s*. 2*d*. and 3*d*. per cubic yard (1·40 and 0·32 Danish Kroner) is obtained, without affecting the reliability of the

concrete; so far as practical, therefore, the standard deviation should be kept as small as possible.

The Relation between Initial Costs and Repair Costs

It has been presumed above that a reliability for the structure of $1-10^{-4}$ is "correct" for the stress level assumed.

The word "correct" should, in this Paper, be understood to mean "economically most advantageous," but the Author has not found generally applicable quantitative investigations for establishing a reliability which ensures that amortization and payment of interest on the initial costs, plus repair-and-maintenance costs, will be the minimum. The following fundamental considerations, from Reference 6, describe a method of reducing capital costs:—

"The probability of failure, to which a building structure is subjected, is obviously an unfavourable circumstance, which also for economical reasons is of great importance, and which even can be calculated in money. If the relative probability of failure R, which so far has been discussed, is multiplied by the total costs K, which a failure will cause, the absolute probability RK thus obtained, expressed in money, can be regarded as a charge, which falls on the building structure. This charge can be reduced in the design of the structure by decreasing the relative probability, but this requires in turn an increase of the initial costs A. In this way the following economical principle for the choice of the permissible relative probability of failure is arrived at: *The relative probability ought to be just so big that the sum of the initial costs and the absolute probability is as small as possible.* This can be expressed as

$$A + R \times K = \text{minimum} \quad . \quad . \quad . \quad . \quad (10)$$

"Here A comprises the total initial costs of the building structure plus the capitalized value of the future maintenance costs, plus demolition costs minus the scrap value. K comprises the total initial costs, plus a certain demolition cost, minus the scrap value, plus all other direct losses occurring by the failure, plus indirect losses resulting from discontinuation of use, etc. Owing to the way in which A appears in the equation, it is, however, only necessary to include the part of A, which is influenced by an alteration of R. Through this a certain simplification is obtained."

An example of this principle of minimizing total annual costs will be found in a Paper [7] by the Author. From the fact that only two failures of floors have been reported in Denmark in the past 30 years, the annual probability of failure for individual floors was estimated at:

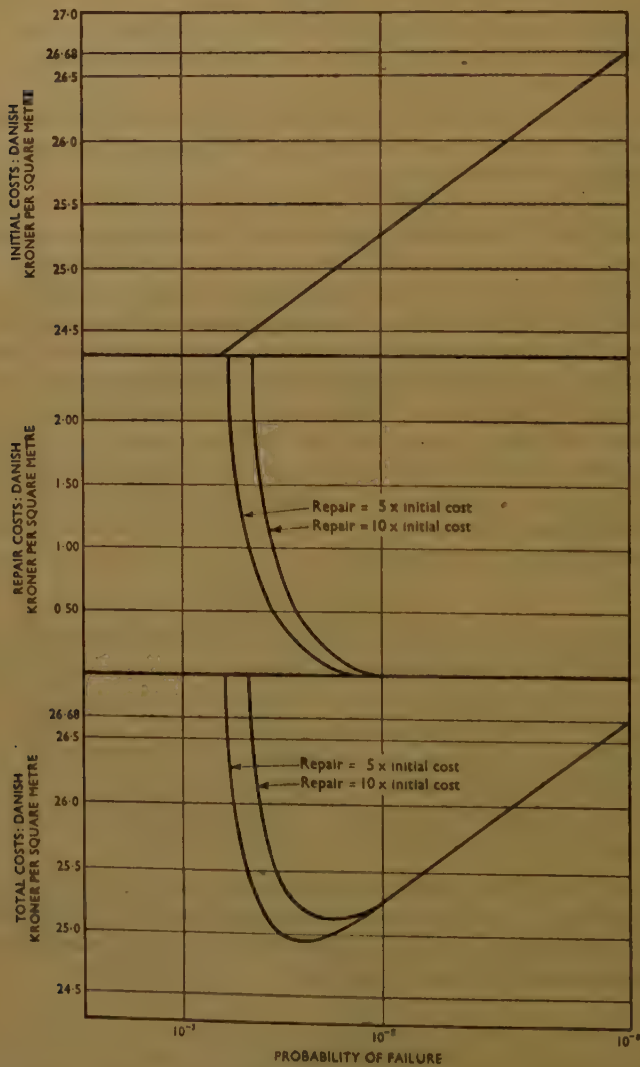
$$\frac{2}{30 \times 5{,}000{,}000} = 10^{-8}$$

the total number of floors being about 5 million.

Accepting this very low figure as a conservative estimate of the

probability of failure for actual floor constructions under prevailing conditions of loading and strength, the economic consequences of increasing the probability of failure were calculated. If the actual loading conditions and the character of the structural materials are unaltered, such an increase may be effected by either lowering the design loads or increasing

Fig 4



INVESTMENT, CAPITALIZED REPAIR COSTS, AND TOTAL COSTS (IN DANISH KRONER PER SQUARE METRE) FOR FLOORS, AS A FUNCTION OF PROBABILITY OF FAILURE

the allowable stresses specified in existing codes for this type of construction.

The economic analysis was carried out in three stages. The upper portion of *Fig. 4* shows the rectilinear decrease of the initial investment, when the probability of failure is increased from 10^{-8} towards 10^{-3}; in the middle is seen the increase in discounted present value of future costs of repair for the same increase in probability of failure—moving from right to left in the Figure. The lower portion shows a vertical summation of the curves in the upper portions, from which it will appear that the optimum probability of failure is about 10^{-4}; in other words the optimum reliability is $1 - 10^{-4}$.

The savings in total investment by increasing the probability of failure from 10^{-8} to 10^{-4} is about 2*d*. per square foot (1·70 kroner per square metre) or 6·5 per cent of the total investment.

Since floors are not exposed to weathering, the only kind of repair cost to be considered is that in respect of damages through failure. In such cases the most advantageous solution is usually reached by increasing the probability of failure, that is, by moving from right to left on the right branch of the curve at the bottom, whereby both components—initial investment and discounted costs of repair—are reduced.

However, most structures are exposed to weathering and will therefore involve heavy maintenance costs. When the total costs of such structures are considered, all future costs of maintenance should be added to the costs of repair and capitalized, and a curve similar to the one at the bottom of *Fig. 4* may be established. The minimum may, however, lie to the right of the present probability of failure. If the principle of minimizing the total costs is applied to such structures, and if the minimum is situated to the right of the present probability of failure, the saving in total costs will entail an increase in initial investment.

Only elucidation of these problems will provide the necessary basis for a rational revision of the standard specifications, with a further classification, and this revision is the necessary condition for an appropriate utilization in practice of one of the most common building materials.

In the Author's opinion, the tendencies described above are so clear that attempts to reduce the standard deviation in practice can, and should, be undertaken without delay; towards this accomplishment, a critical examination is given of the various phases in the practical production and control of concrete.

Regulation and Improvement of Concrete Homogeneity in Practice

Before considering the various methods of improving the concreting methods and the control in practice, it may be of interest briefly to study the causes of the variations.

The Causes of the Variations

Whilst the imperfection of the concrete-mixers may be understood and accepted, it is perhaps astonishing that each new batch of concrete will not give the same results as the previous one. This has various causes, which are illustrated in the left half of Table 4.

TABLE 4

Component of variation		Constructions			Test specimens		
No.		Origin	Contributes to δ_w	Contributes to ω	Origin	Contributes to δ_w	Contributes to ω
1	Concrete	Cement		x	Cement		x
2	Concrete	Gravel		x	Gravel		x
3	Concrete	Weighing		x	Weighing		x
4		Mixing	x		Mixing	(x)	
5	Execution	Transport	x	x	(Transport)	(x)	
6	Execution	Compaction	x	x	can be standardized		
7	Execution	Storing	x	x			
8	Testing	—			Sampling		
9	Testing	—			Testing		

Items 1, 2, 3, and 4 originate, as will be seen, from the concrete production proper, until the moment it leaves the mixer. By means of production control it will be possible to supervise these variations continuously.

Items 5, 6, and 7 originate from the other working site processes, and it is only possible to control them visually during the concreting. They can only be controlled qualitatively by supervision and can only be determined quantitatively by tests on the completed structure.

Control and Regulation in Practice

The production and the quality control of concrete in practice may, for instance, be divided into the following groups according to the relation of the control to the different parts of the working process :—

(1) Manufacture and control of the concrete constituents.
(2) Trials with the materials available.

(3) Mixing, placing, compacting, curing, etc., of the concrete.
(4) Supervision of the execution of the work.
(5) Control of the quality of the completed structure.

It will be noted that two fields of control have been included which normally do not apply to other building materials, namely, groups Nos 2 and 3. This is because the concrete is produced on the building site proper.

The five groups will be dealt with individually in the following.

(1) *Manufacture and Control of the Concrete Constituents*

The materials to be used for a concrete structure must be subjected to various physical and chemical investigations, partly individually and partly as components of an otherwise standard concrete or mortar, in order to enable the investigator to form a first impression of their suitability.

After this first investigation, on which the subsequent proportioning and trials are based, the materials must be examined continuously to ensure that their quality is maintained.

Cement.—Considering the large variations which are often experienced in the quality of the cement when it is used on the working sites, it would perhaps be appropriate if attempts were made to find the causes of these variations and, when these causes had been explained, to improve the homogeneity.

An example of the great variations occurring in the quality of the cement was to be seen when, during the construction of the runways at Heathrow, *mortar* cubes were cast continuously, and it was found that the coefficient of variation amounted to about 8·6 per cent; on the basis of the variation of the *concrete* cubes made simultaneously, it was estimated that 48 per cent of the variation of the concrete cubes was caused by variations in the quality of the cement.

So far as the Author can see, this estimate is, however, hardly correct. If due consideration is taken of the big dispersion in the other variables occurring at the same time, only about 20 per cent of the total compressive strength variation may, with certainty, be traced to the cement.

Whether variations of this magnitude always occur is not sufficiently clear, but it is natural and not difficult to start an investigation in this field, considering that there seems to be an actual possibility of reducing the great variation in the concrete quality; it must be hoped that the cement industry will readily collaborate with the cement users to find the most economical combination of variation and cost of production.

Aggregates.—Most specifications establish limits for the permissible quantity of over-sizes and sub-sizes of the different sand and stone fractions. But very frequently these tolerances are not being observed in practice.

Precise information on the extra costs resulting from strict maintenance of the prescribed tolerances is at present not available. However, there is reason to believe that these costs will be relatively small compared with the savings in cement which may result from the greater uniformity of the grading, and it is to be hoped that the aggregate manufacturers will look carefully into this question.

It would be desirable to have reliable statistical control methods, allowing intervention in the supplies in due time. Until a greater knowledge of the size of the variations actually occurring in the grading is acquired, the necessary statistical background for the adaptation of such methods is not obtainable.

(2) *Trials with the Materials Available*

From an economical point of view, it is, of course, very important that the results of these trials reflect as closely as possible the actual properties in the structural members, and the following considerations aim at throwing some light on this question.

Even if the trial-test specimens are cast in the same way, using the same materials and equipment as the completed structure, there will be a risk that the concrete which is later produced will not be exactly the same.

The reasons for this risk are :—

(*a*) The same reasons which cause differences between control-specimens and the actual structural members. From Table 4 it thus appears that whilst the components of variation Nos 1, 2, 3, and 4 (and 5) will generally act in the same way in the test specimens as in the completed structure, Nos 6 and 7, which cannot be standardized, will in practice cause differences between the quality of the test specimens and the completed structure. During the sampling and the testing proper, further apparent variations will be introduced, which do not occur in the construction proper (Nos 8 and 9).

(*b*) That during the actual construction period, changes in the materials and the external conditions—which do not present themselves during the short trial period—will slowly occur.

Since the differences between test specimens and the actual structure are not the same for average strength and standard deviation, these two quality-characteristics are dealt with separately in the following.

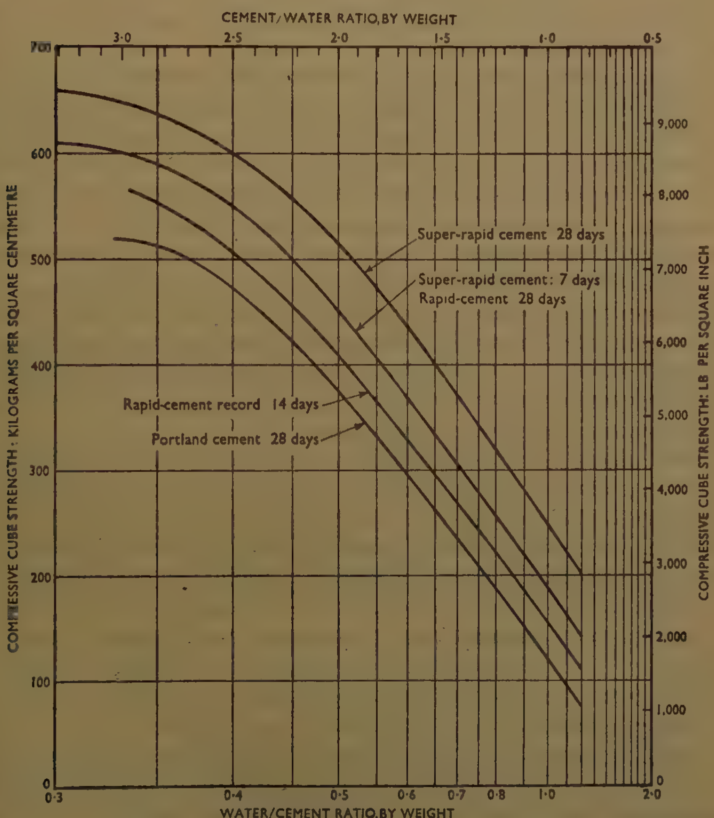
Standard deviation.—As will appear from the above, it is not advisable at the present stage of the technique to draw conclusions as to the variations in the building constructions from the variations in the trial- and control-test specimens, and it is therefore hardly necessary to determine the strength of the concrete cubes in the hope of making a judgement of the variation.

In practice, during the designing and the first stages of the work, it will therefore be necessary to judge the variation by experience from previous structures of the same kind and executed in the same way.

Average strength.—The same causes which prevent an estimation to be made, from the results of trial concretings, of the variation of the quality in the building construction, make it impossible to estimate the average value of, for instance, the strength—perhaps especially because of changes of the materials (mainly the cement) in the course of time.

Until measurements on the completed structure can be made, and this may often last many weeks or months, the best judgement of the average value of the strength may, in the Author's opinion, be obtained from curves (as, for instance, *Fig. 5*), in so far as they are based on a sufficiently great

Fig 5



TYPICAL RELATIONS BETWEEN CONCRETE CUBE STRENGTHS AT 7 AND 28 DAYS AND WATER/CEMENT RATIO FOR THREE TYPES OF DANISH CEMENTS

number of measurements of strengths actually reached in completed structures.

The correctness of this judgement depends—so long as the cement works do not change their methods of production—mainly on the attainment of the degree of compaction assumed in the determination of the curves.

(3) *Mixing, Placing, Compacting, Curing, Etc., of the Concrete*

The first difficulty encountered in control during construction, using the present technique, is that the test results will be available so late that the conditions under which they were made will often no longer exist.

In *Fig. 2* it will be seen that, even with the employment of 7-day results, a curious situation will often occur, namely, that either the quality has in the meantime improved by itself, for which reason a search for the sources of error will be useless, or it will appear that the quality is excellent, whereas it is in fact bad.

A more frequent control of the properties of the concrete prior to setting has been advocated in recent years, despite the difficulties which the employment of even 7-day strengths will cause; also recommended is the control of the grading of the aggregate and the water content of the sand, in the hope that such a control would be sufficiently rapid to secure a more uniform concrete quality. Such control methods have been in use for some years on several big sites, but it must be admitted that even those are not fitted for regulating the production.

Under awkward conditions, the moisture in the sand will vary so rapidly that even the most energetic control cannot keep pace unless completely continuous methods are invented, which would also be applicable on medium-size and small working sites. Exactly the same conditions apply to the control of the grading of the gravel.

Furthermore, when it is realized that neither the determination of the moisture content nor the sieve analyses will generally be fully understood by the man who is to make them, and that these control methods are, on the whole, very unpopular in practice, it would be wise to think twice before recommending further this kind of working site control, and to try other ways.

The two most essential items to control are the cement and water contents.

The cement content may be efficiently controlled by inspection, and checked by counting the empty cement bags every day.

Regarding the water content, the difficulty is mainly the control of it in the sand, and this cannot be done in a satisfactory way by means of the discontinuous methods. It has therefore been tried to control the water content by means of the consistency, either by continuous mechanical appliances or by inspection. So long as the grading of the

solids is kept constant, this method has proved to be satisfactory [8], but unfortunately this is not normally possible.

The continuous control of the aggregate grading by means of sieve analyses is the first step in an attempt to keep the grading constant, and the next step should be that changes of the composition of the aggregate are made, when changes in the screenings of the raw materials are ascertained.

However, this sieve control as mentioned is encountering difficulties in practice and is subject to some delay, and since the necessary further corrections have proved to be rather impracticable and uneconomical on the working sites, it will no doubt be necessary, as part of the effective minimizing of the total costs, to have the proportioning made—strictly to requirements—by the suppliers.

It is common practice to-day that the coarse aggregate is delivered in two or three fractions, whereas the sand is often delivered in one fraction only. This is inexpedient, because the sand calls for about half the total quantity of water, whilst the coarse aggregate calls for less than one fourth of the quantity of water. Much is to be said, therefore, in favour of further fractioning of the sand but, since the economic aspect of the sand-fractioning is important, the problem should be given much consideration.

During these considerations it should be remembered that, at the present stage of the technique, the decisive factor is not the maintenance of more or less hardly definable ideal gradings, but that the grading (good or less good) available is kept constant so far as possible.

For instance, a straightening out of the ordinary S-shaped curve will, for the producer, inevitably involve a rather considerable waste of the medium fraction, which he will only be paid for by raising the price for the quantity which he is actually selling. This increase in price will perhaps not always be compensated by an improvement of the quality or a saving of the cement.

If, eventually, the aggregate grading on the working site is kept constant, the main control may be directed to the judgement of consistency, and it will be reasonable to give this greater attention than so far.

On small working sites it will probably be necessary to rely upon visual inspection by the man at the mixer, but this control must frequently be verified with objective determinations of consistency.

Such verification must take place by one of the existing mechanical methods, among which should be mentioned the common slump test,[9] which is a sensitive measure of the plastic consistencies, and the Vebe apparatus,[10] which is sensitive to dry consistencies.

These and numerous other methods, which have been invented from time to time, suffer from a number of shortcomings both with regard to definition and methods of measurement, as well as to their ability to supply

results which—over the whole range of water contents—approximate to the actual placeability of the concrete.

Many of the methods have been examined by the Author, and further details may be obtained from Reference 8.

On large working sites it will probably be appropriate to use consistency meters of the wattmeter type [3] for the control of the consistency by means of the power consumption of the mixer. Brief information and a number of preliminary results regarding these methods may also be found in Reference 8.

This simplified method for keeping the quality homogeneous has been tested on a large scale at Heathrow, and it has been possible to reduce the coefficient of variation within each batch to 5 per cent, and from batch to batch to 11 per cent.

(4) *Supervision of the Execution of the Work*

The control just mentioned concerns only the correct proportioning and manufacturing of the concrete.

During transport, pouring, compaction, and storing, a series of new sources of errors are introduced, which can be ascertained only by measurements of the completed structure, and these measurements will, under all circumstances, be available so late that they cannot be used for regulation of the production.

To make up for this, the owner, as well as the contractor, will have special superintendents to supervise the execution of the work. It means that control of the working technique, which is assumed in the specifications and for the proportioning, is maintained. Even if the judgement of the superintendents is to a great extent only qualitative, the importance of it must not be underestimated for the following reasons:—

(*a*) The superintendents will intervene when the work is carried out in an unsatisfactory way.

(*b*) It is general experience that the mere presence of such a control, facilitating the exposure of a fraudulent intent and the substantiation of complaints of gross errors, will effect an improvement of the quality.

(*c*) The awareness of the presence of the superintendents will induce all parties to carry out the work in a way that is in accordance with the assumptions just mentioned.

(5) *Control of the Quality of the Completed Structure*

For economical reasons on most working sites, it will only be possible to control the quality of the structure by means of small test specimens cast independently.

As previously mentioned, this method is barely sufficient—compare

also Table 4—even if it provides the possibility of using exactly the same materials as those of the structure. It is highly desirable that a thorough research within this field is undertaken, since it will appear from the foregoing that the conversions used at present between the average strength of the test specimens and the actual average strength of the completed structure, as well as between the standard deviations of these, are based upon rather incomplete data. Considering the large quantities of concrete cast, it will be seen that a unilateral error of only a few per cent (which probably is greater) will mean considerable loss to the building trade.[11]

The above-mentioned difficulties, involved in judging the quality of the completed structure from the test specimens, actually make the last part of the site control rather delusive. These facts were especially referred to in the introduction, where it was mentioned that the strengths required in standards and specifications to ensure the minimizing of the initial and maintenance costs are very vaguely based. As regards all the other problems of control, it has been endeavoured, and with some success, to lay down well reasoned rules for the practice followed, but as to this point it must be admitted that there is still considerable ignorance.

Considering the many difficulties, briefly described in the foregoing, and on account of the economic obstacles for testing the entire structure, it is not surprising that this last phase of the control is often very insufficient as regards representativeness, and the consequences to be drawn from too bad a quality is thus generally very vague. So far as the Author can see, the solution of the problem calls for a development of non-destructive testing methods, as for instance the wave-velocity[12] or gamma-ray methods, which may give an impression of the conditions in the completed structure of which it is not possible to form a reliable picture by means of test specimens.

Acknowledgement

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The Paper is accompanied by eight sheets of diagrams, from some of which the Figures in the text have been prepared, and by the following Appendix.

Appendix

The simplified example given in connexion with *Fig. 3*, according to which it should be possible to reduce the mean compressive strength in a construction by a considerable amount (if the standard deviation of compressive strength is reduced), rests on the following assumptions:—

(1) The frequency distribution of compressive strengths of test specimens is the normal (Gaussian) distribution. Other forms of distribution relating to the testing of materials have been considered recently, see references 13 and 14.

(2) The parameters, mean and standard deviation, of the distribution, are known. In practice there are only estimates derived from a limited series of test values.

(3) The compressive strengths of conceivable test specimens from the completed structure will be distributed in the same way as the 28-day strength of the test specimens actually sampled during the period of construction. This assumption is discussed in this Paper in relation to Table 4.

(4) The frequency distribution of compressive strength in the completed structure is independent of the dimensions of the structural member. From experiences with the testing of materials, it is, however, well known that the form and parameters of the frequency distribution vary with different shapes and dimensions of the test specimen.

Further references on this point are given in the Bibliography; see for instance references 6, and 14 to 28 inclusive. So far as the Author knows, it has not been possible to establish a relation between the frequency

distribution of compressive strength of concrete test specimens with given dimensions, and the frequency distribution for structural members made of identical concrete but with other dimensions.

(5) The only type of failure considered is that caused by loads, whereby the compressive strength of a member of the construction is exceeded. Theoretically it is possible to combine the risks of failure from other causes (fatigue, instability, etc.) when the joint frequency distribution of the relevant properties of the material, and the corresponding maximum influences (compare (6) below) are known.[6, 29]

(6) The maximum level of stress in all parts of the construction is uniform and known. This assumption again implies that :—

(*a*) The structural members are correctly dimensioned according to an adequate theory.[29, 30]

(*b*) The maximum loading conditions of the structure are known.

If no maximum loading conditions can be specified apart from a frequency distribution of loads, it is possible to design the members in such a way that the risk of failure can be given a value to be fixed in advance, for instance 10^{-4}, by combining the distribution of stress derived from the distribution of the loads with the distribution of resistance (compressive strength) of the concrete. Suppose that the calculation of risk of failure is based upon random variations in the quality of the concrete as well as on random variations in the load about a mean, $\bar{P}$. Then an improvement of the concrete quality (reduction of the standard deviation on the strength) will give a smaller possibility of lowering the mean value (strength or dimensions) than if the risk of failure had been calculated with a view to $\bar{P}$ as a maximum loading, provided that the same risk of failure is maintained. Contributions to the solution of this problem can be found in references 6 and 29.

(7) Only plain concrete structures are considered. Further complications are introduced when reinforced structures are considered. For instance, the risk of failure caused by excess of the yield limit of the reinforcement in the zones of tension ought to be combined with the risks of failure in the zones of compression, as considered above.

Paper No. 5915

"Entrained Air in Concrete"*

by

Peter Joseph Frederick Wright, B.Sc.

(*Ordered by the Council to be published with written discussion*)†

SYNOPSIS

The term "entrained air" refers to air intentionally incorporated, by means of an admixture, in concrete in the form of numerous microscopic bubbles. The presence of this air improves the workability of the concrete, and greatly increases its resistance to attack by frost. The latter effect has been responsible for the widespread adoption of the process in the United States of America. On the other hand, the entrained air causes a reduction in the strength of concrete of any particular composition. The improvement in workability, however, enables the water content to be reduced, and for a given workability no loss in strength need result. In addition to these effects, the tendency for fresh concrete to bleed is greatly reduced, water absorption of the set concrete is reduced and, as a result, the resistance to chemical attack is slightly improved. A relatively high degree of control is necessary if the process is to be used effectively.

A wide range of organic materials can be used as air-entraining agents, the most important groups being resins and wetting agents. Various proprietary materials have been developed for the purpose and certain of these, of American origin, are available in Great Britain.

The amount of air entrained in a concrete mix varies not only with the amount and type of admixture used, but also with factors such as the grading of the aggregate, the workability of the mix, character of the mixing, and the temperature. Considerable variations in air content are experienced in practice, and it is therefore necessary to determine the air content at regular intervals and adjust the amount of air-entraining agent accordingly. Three methods of determining the air content are described and their relative merits discussed.

The Paper also includes a suggested method for modifying the process of mix design to allow for the effect of entrained air. The procedure is to substitute the air for such quantities of water and aggregate that the workability will remain constant for a given cement content.

It is not considered likely that air-entrainment will prove to be an asset in the construction of concrete roads of high quality in Great Britain. Roads which are not of high quality (for example, many housing-estate roads) are often constructed by contractors inexperienced in this class of work, and without adequate supervision and control. The inclusion of an air-entraining agent in such cases may help to prevent the consequences of using such methods by reducing laitance and improving durability.

In structural work the advantages of air-entrainment are partly offset by the need for an additional operation in introducing the correct quantity of air-entraining agent into the concrete, and by the need for a greater degree of control than is at present exercised on many contracts. On many minor jobs, however, where strength is not of primary importance, air-entrainment may help in enabling full compaction to be

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achieved without segregation or excessive laitance. This applies also to precast products such as kerbs, where a smooth finish is desired, and which are frequently liable to be damaged by frost as a consequence of using too wet a mix.

Introduction

It is the aim of this Paper to review, in detail, the effects of air-entrainment in concrete so that engineers may be better able to assess its advantages and disadvantages in particular circumstances. During the past 12 years the principle of entraining air in concrete has become increasingly popular in the United States of America as a means of reducing, or eliminating, the scaling of concrete roads and other structures by the action of frost—frequently accentuated in the case of roads by the use of de-icing chemicals. Incorporated in the mix is a very small amount of a material which, during the mixing process, entrains air to the extent of about 5 per cent by volume of the concrete in the form of myriads of microscopic bubbles distributed throughout the mass. This entrained air modifies the properties of the concrete in both its plastic and its hardened state in a number of ways. In the plastic state, the concrete becomes more workable, and is reputed to be less subject to segregation and bleeding. In the hardened state, the concrete is much more resistant to the effect of frost but, at the same time, suffers a reduction in strength; this latter effect can be overcome almost entirely by reducing the sand proportion and water/cement ratio of the mix to restore the workability to its former value.

Although air-entrained concrete has become almost universally used for road work in America, its popularity in Great Britain has not spread in a similar manner. This is probably caused largely by the fact that frost-scaling of concrete roads is not a serious problem in Great Britain because of the drier mixes used, and because of milder winter conditions involving only an occasional use of de-icing chemicals. There would therefore generally be no marked advantage accruing from the use of air-entrained concrete for concrete roads where the mix is sufficiently dry to be laid by mechanical methods. It may be suggested that the greater workability might be an advantage in helping to produce a good riding quality, but in practice it has been found that this is not the case except on level roads. Even on small gradients, concrete of higher workability than that normally used in roads compacted by machinery tends to flow downhill when it is vibrated, and a poorer riding quality results. The construction of concrete roads of high quality, however, is a specialized type of work calling for expensive equipment and experience in the technique. In many cases, for example on housing estates, roads are laid by small contractors who have neither the equipment nor the experience to produce concrete roads of high quality and who compact the concrete manually. Under such circumstances, and especially where the grading of the aggregate leaves

much to be desired, perhaps the most important problem is to obtain a fully compacted concrete without segregation or excessive laitance. In this case, air-entrainment may be of considerable assistance since it increases the plasticity and cohesion of the mix, and reduces the tendency for laitance to occur with overworking of a rather wet mix, and for bleeding to take place.

In constructional and other uses of concrete, air-entrainment has been used to a considerable extent in America, partly to improve the durability of the concrete and partly in virtue of the greater workability. It has not, however, become widely used in Great Britain, and in many classes of work any advantage which might result is offset by the need for an additional operation in incorporating the correct quantity of air-entraining agent in the concrete, and for the extra degree of control which is necessary. Without adequate control it is possible for the air content, and hence both the strength and workability, to vary considerably.

Certain types of precast products, such as kerbs and fence posts, are frequently used in exposed positions and are liable to be damaged by frost. In products such as these, strength is not of primary importance and a smooth finish free from voids and honeycombing is one of the features often desired by the purchasers. Under such circumstances the use of air-entrained concrete may result in an acceptable and more satisfactory product being economically produced. In many minor jobs, too, where ease of placing and finishing is more important than strength—such as in pipe haunchings—air-entrainment may prove to be an asset.

The History of Air-Entrainment

During the 1930's it was observed in the north-eastern States of America that certain stretches of road, in which a blend of Portland cement and "natural" cement was used, were standing up remarkably well to the effects of frost and the use of calcium chloride for the removal of snow and ice, while other roads were suffering seriously from scaling.[1] The difference was so noticeable that an investigation was started into the underlying causes and the records pertaining to the laying of the roads were subjected to close scrutiny. Two important factors were disclosed. First, the more durable roads were associated with lower densities of concrete—a surprising fact in view of the common knowledge that concrete must be dense if it is to be strong and durable. Secondly, the natural cement used in the more durable roads was obtained from mills where a small amount of beef tallow had been added to the cement as a grinding aid. Further investigation showed that the tallow or other grinding aid such as resin or, in some cases, lubricating oil which had leaked from the bearings of the grinding mills, was responsible for entraining in the concrete the air

[1] The references are given on p. 357.

which produced the various modifications in its properties. Since then, interest in the effect has grown rapidly. An extensive amount of experimental work has been carried out in America, new tests and apparatus have been devised, and a great deal has been written about the subject. In many of the States, air-entrained concrete is now used for all road work, and to a considerable extent for structural work.

The Effects of Entrained Air on the Properties of Concrete

The three principal effects of air-entrainment are as follows :—

(1) An improvement in workability.
(2) A modification in pore structure leading to greater durability.
(3) A reduction in strength.

The Improvement in Workability

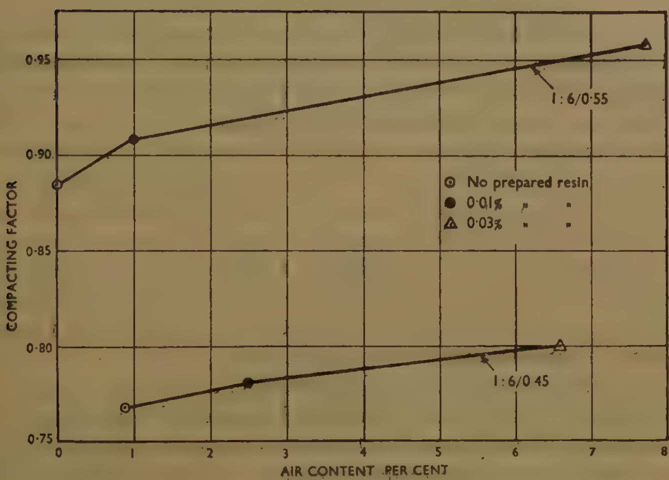
The entrainment of air improves the workability of concrete to a marked extent. Five per cent of air may increase the compacting factor [2] by 0·07, or the slump from ½-inch to 2 inches, although the actual increase depends on various factors besides the amount of air entrained ; *Figs 1* show some typical values obtained at the Road Research Laboratory. The increase in workability is rather greater for the wetter mixes than for the drier ones, and for the leaner than for the richer ones. American work [3] suggests also that the increase is greater for angular aggregate than for rounded ones.

The improvement in workability has not been satisfactorily explained but is probably caused by the differences between the rheological properties of the water-cement-air mixture and those of a normal water-cement mixture. This aspect has not been widely studied, and the effect has been attributed to the behaviour of the air bubbles as particles of fine aggregate, which are elastic and have negligible surface friction. This would account for the increased workability, and also for the over-sanded appearance of a normal mix which has had an air-entraining agent added without making any adjustment to the mix proportions. The percentage of sand in the aggregate may be reduced to counteract this latter effect, and advantage should be taken of this possible reduction, for it allows the water content to be reduced.

In addition to the improvement in workability as measured by the compacting factor, slump, or other normal test, the entrained air modifies the character of the fresh concrete in a way which is not easily described and for which no test has so far been developed. The concrete containing entrained air is more plastic and more easily handled than ordinary concrete, and is preferred by labourers working with it.

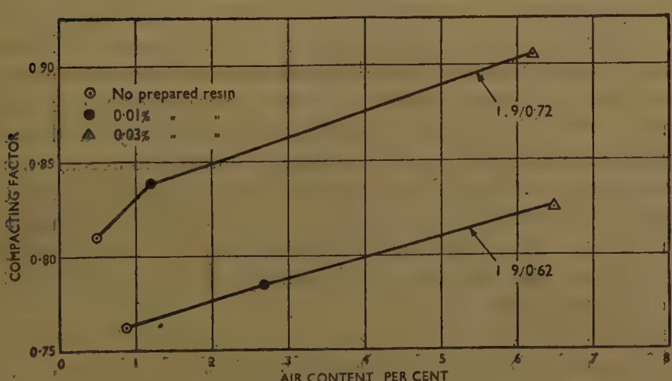
Figs 1

(a)



EFFECT OF ENTRAINED AIR ON WORKABILITY OF RICH MIXES

(b)



EFFECT OF ENTRAINED AIR ON WORKABILITY OF LEAN MIXES

The Modification in Pore Structure

The modification in pore structure produced by air-entrainment first manifests itself as a reduction in " bleeding," that is, in the formation of a layer of water on the surface of the concrete after compaction and finishing. This phenomenon is more pronounced and more serious in the laying of roads and other flat slabs than in structural work. Bleeding is caused by the settlement of solid particles which are initially separated from each other by films of water ; as the particles settle, the water below them is able to flow outwards and then upwards around them. In its passage to the surface this water leaves capillaries which later allow water to penetrate the hardened concrete, and so increase the damaging effect of frost. Air bubbles, on the other hand, are maintained in an approximately spherical shape by the surface tension. These bubbles are very stable and can withstand considerable pressures ; they are unable to pass through the interstices between the particles as easily as water, but remain in position and keep the solid particles apart ; the water is not forced out from under the particles and bleeding is reduced.

This reduction produced by air-entrainment has been reported by various authors [4, 5] in America, where it has enabled the surface of the road slabs to be finished immediately after compaction instead of waiting an hour or more as was the former practice. The reduction in bleeding generally accompanies a reduction in laitance, which also contributes to improved frost-resistance. In Great Britain, where drier mixes are used for road work, bleeding and laitance are not normally experienced, but in structural work the mixes generally have to be more workable, and here bleeding may be reduced by the use of air-entrained concrete.

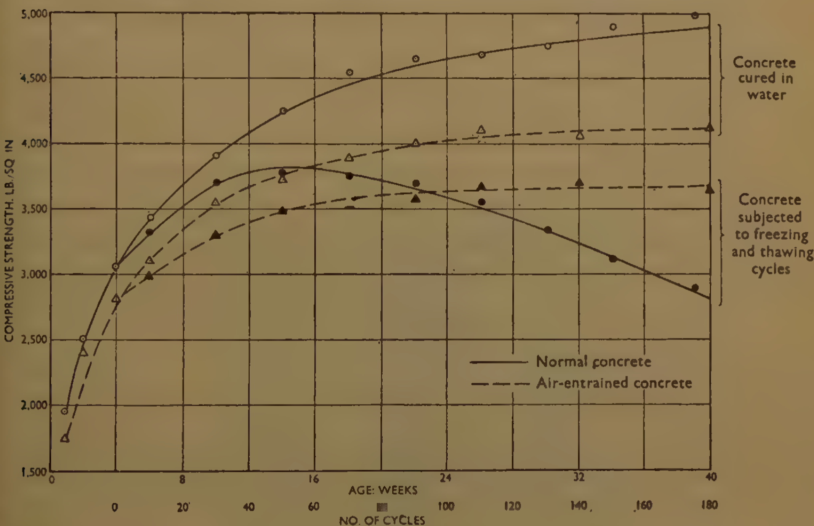
The modification in pore structure is believed to be responsible for the marked improvement in resistance to frost attack, which was the primary reason for the adoption of air-entrainment in America. A hypothesis regarding the mechanism of failure due to the action of frost was evolved by Collins in 1944.[6] The mechanism described depended on the existence in the concrete of voids of various sizes interconnected by capillaries, the latter being largely formed by the bleeding. Whilst the actual volume of voids in air-entrained concrete is greater than in ordinary concrete, the entrained air is in the form of minute discrete bubbles of comparatively uniform size and regular spherical shape and it is not easily replaced by water. Further, the reduction in bleeding produces a reduction in the extent of the capillaries.

These differences are believed to reduce the tendency for large crystals of ice to form in the concrete, thus decreasing greatly the chance of damage. It has also been suggested that the bubbles of air provide a reservoir for some of the expansion which will necessarily occur on freezing when the water voids are saturated.

The greater resistance to damage by frost has been demonstrated by full-scale, and laboratory tests in America. No full-scale comparative

tests have been carried out in Great Britain, but tests at the Road Research Laboratory using daily cycles of freezing and thawing have shown the superiority of the air-entrained concrete. The results of these tests are shown in *Fig. 2*. It has already been pointed out that frost damage to road slabs does not present a serious problem in Great Britain. In *Fig. 3*, however, a precast kerb is shown which has been damaged by the action of frost, and is typical of many that may be seen.[7] In fields such as this the use of an air-entraining agent may prove to be an economical method of obtaining the required durability.

Fig. 2



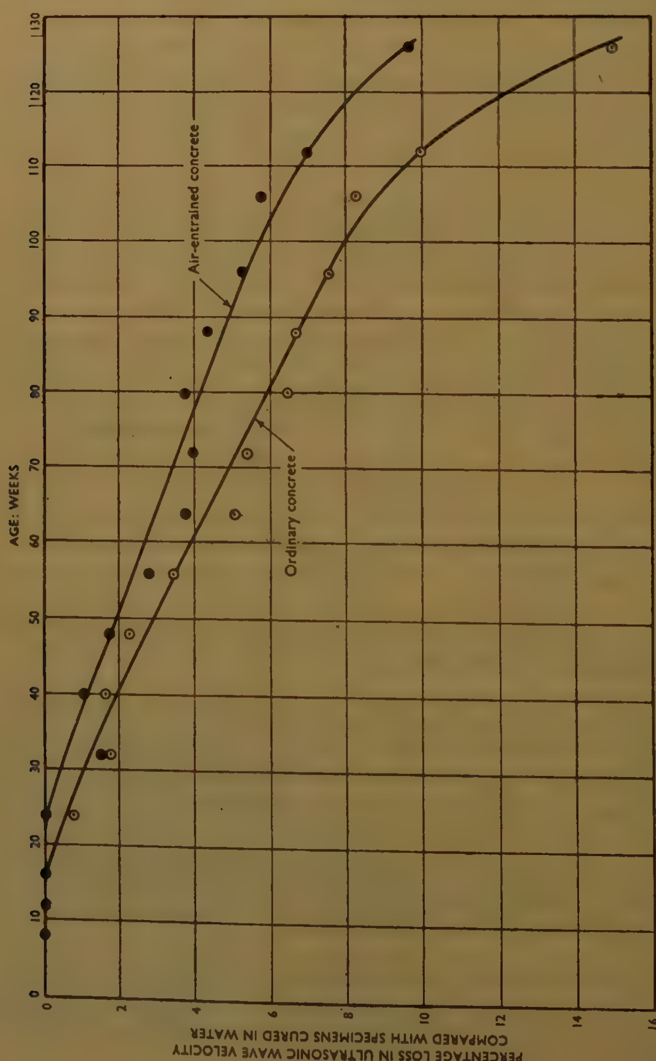
THE EFFECT OF FREEZING AND THAWING CYCLES ON THE COMPRESSIVE STRENGTH OF NORMAL CONCRETE, AND CONCRETE WITH ENTRAINED AIR

A minor effect of the modified pore structure is a reduction in permeability and absorption. This effect has not been studied in detail but tests on samples of two comparable concretes made in the Laboratory, one with entrained air and one without, gave average values of water absorption of 1·2 per cent and 1·7 per cent respectively. This indicates that whilst air-entrainment may not provide a very effective method of producing a concrete of high impermeability, an air-entrained mix will certainly be no worse than ordinary concrete in this respect and will probably be better.

In view of the lower permeability and absorption, it seemed probable that the resistance of air-entrained concrete to chemical attack might also be greater than that of normal concrete, and tests are in progress at the

Road Research Laboratory to determine whether or not this is so. Specimens of comparable mixes of ordinary and air-entrained concrete have been immersed in a 5-per-cent solution of magnesium sulphate, and the deterioration in quality has been assessed by measuring the decrease in the velocity of an ultrasonic wave through the specimens, by a method developed at the Road Research Laboratory.[8] The effect of the entrained air in reducing the damage caused by chemical attack has not been so marked as its effect in reducing damage caused by frost, but the results so far obtained show a

Fig. 4



DETERIORATION OF ORDINARY AND AIR-ENTRAINED CONCRETE IN A SULPHATE SOLUTION

Fig. 3

TYPICAL FAILURE OF A CONCRETE KERB CAUSED BY THE ACTION OF FROST

Fig. 8

PRESSURE-TYPE AIR-METER FOR DETERMINING THE AIR CONTENT IN AIR-ENTRAINED CONCRETE

small, though significant, improvement with the entrained air. The results are shown in *Fig. 4*.

It appears likely that the lower permeability and greater resistance to chemical attack result partly from the lower water content employed in air-entrained concrete and partly from the modification of the pore structure. Unfortunately it has not been possible to isolate these effects, since the air content and the water content cannot be varied independently without making alterations in either the aggregate/cement ratio or the workability of the mix.

The reduction in bleeding suggests that a reduction might also be observed in the tendency for segregation to take place during working of the concrete. It has been claimed in America [9] that a reduction has in fact been observed in the segregation of fresh concrete during transport. This possibility has attracted particular attention in the ready-mixed concrete industry, and where concrete is mixed at a central plant on a large site and conveyed by dumper. Quantitative tests, however, are not easy to carry out satisfactorily and the few which have been made have not been encouraging.

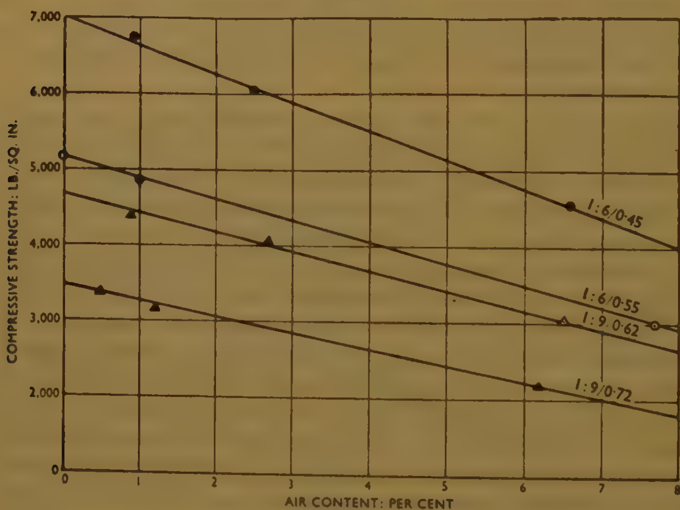
Kennedy [5] studied the segregation produced during compaction by carrying out some tests on hardened concrete using resistance to abrasion as the criterion of quality in different portions of the specimen, and these showed greater variation in the specimens containing entrained air than in those of normal concrete. On the other hand, he leaves the matter in doubt by quoting some figures for compressive strengths at various depths up to 32 inches in ordinary and air-entrained concrete, which showed greater variation in the ordinary concrete. Some tests have been made at the Road Research Laboratory, using the percentage of coarse aggregate and the velocity of transmission of an ultrasonic wave as criteria of quality at various depths in 6-inch-by-12-inch cylinders subjected to excessive vibration on a vibrating table. These tests have not been exhaustive, but the general indication is that the variation in quality from top to bottom is more pronounced when entrained air is used, though there is less tendency for a thick layer of mortar to be brought to the top by overworking.

In road work, where comparatively dry mixes are used, segregation during compaction does not present so great a problem as segregation during handling, and in particular during discharge from the mixer or from the dumper or lorry on the formation. An improvement in this respect might be anticipated in view of the greater cohesiveness which appears to be characteristic of air-entrained concrete. Tests have been carried out at the Road Research Laboratory using a sloping chute, and these have indicated that, in the case of a well designed mix having only a slight tendency to segregate, the entrainment of air yields no improvement, but a mix which segregates considerably is improved by air-entrainment.

The Reduction in Strength

The inclusion of air in concrete having a given water/cement ratio results in a loss of strength, whether the air is intentionally entrained or is present as a result of incomplete compaction, but the use of air-entrainment has become almost universal in the United States of America. It has been found by experience there that when the process is properly applied little or no loss of strength need result, and it is possible that under certain circumstances a gain in strength may be achieved. It is true that, for a given water/cement ratio, an increase in air content results in a loss of strength, but the entrainment of air enables the water/cement ratio and the sand content to be reduced substantially, thereby regaining most if not all of the lost strength.

Fig. 5

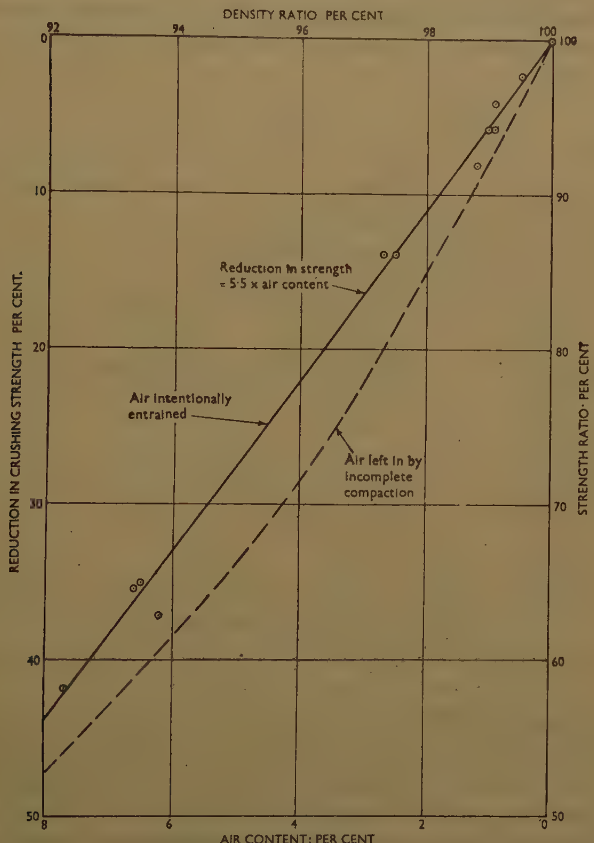


EFFECTS OF ENTRAINED AIR ON COMPRESSIVE STRENGTH

The results of tests on the compressive strength of air-entrained concrete carried out at the Road Research Laboratory are shown in *Fig. 5* and *Fig. 6*. In these tests four mixes were investigated and the air content was increased without making any other adjustment to the mix proportions; *Fig. 5* shows the actual strengths obtained, and it will be seen that the strength decreases in proportion to the amount of air. In *Fig. 6* the same results are shown expressed as a percentage of the strengths of concrete containing no air; the latter were estimated from *Fig. 5*. It will be seen that a single straight line is obtained, this representing a decrease in strength of 5·5 per cent for each 1 per cent of air. Also plotted is a portion of a curve for the strength of concrete containing air voids resulting

from incomplete compaction,[2] and this shows that the effect on the strength is materially the same whether the air is entrained intentionally in the form of numerous minute bubbles, or occurs unintentionally in the form of comparatively large irregular voids. It must be appreciated, however, that the air voids caused by incomplete compaction do not provide the other effects of entrained air, such as increased durability and workability.

Fig. 6



EFFECT OF INCLUDED AIR ON THE COMPRESSIVE STRENGTH OF CONCRETE

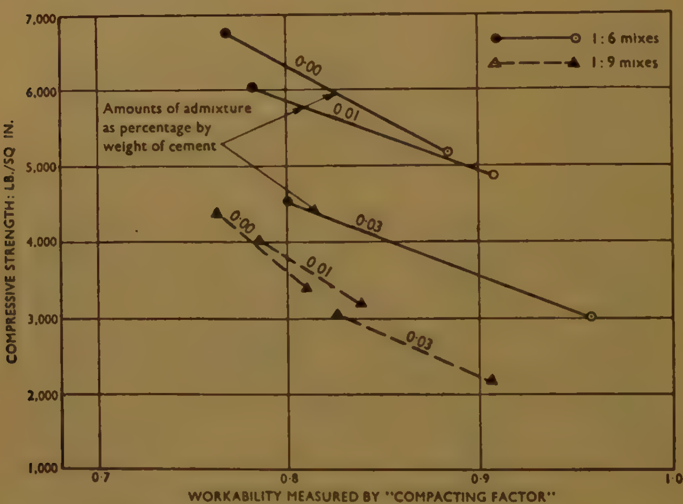
The results shown are for compressive strength only, but American work [4] indicates that the decrease in flexural strength is rather less.

Combined Effect on Strength and Workability

The effects of entrained air on workability and on strength for a concrete with a given water/cement ratio have been discussed, but in

practice it is the relationship between workability and strength for a varying water/cement ratio which is important to the engineer. The entrainment of air improves workability while reducing strength. A reduction in water content has the reverse effect, and if the workability remains constant the loss in strength, if any, will be small. The results previously shown in *Figs 5* and *6* have been plotted in a different manner in *Fig. 7*, to show whether a higher or lower strength for a given workability will be obtained by using entrained air. This diagram (*Fig. 7*) shows the relationship between compacting factor and compressive strength for different amounts of air-entraining agent, expressed as a percentage by weight of the cement.

Fig. 7



STRENGTH/WORKABILITY RELATIONSHIP FOR CONCRETE CONTAINING ENTRAINED AIR

Considering the 1 : 6 mixes, if for example a compacting factor of 0·8 is required, the strength of plain concrete will be 6,300 lb. per square inch, whilst that of concrete containing 0·03 per cent of air-entraining agent (6·6 per cent air) would have a strength of only 4,540 lb. per square inch.

In the case of the 1 : 9 mixes, there is no significant difference between the three lines shown, and it seems likely that for mixes leaner than 1 : 9 by weight, the entrainment of air would result in a definite increase in the strength for a given workability, and vice versa. This has also been indicated by American work.[10] The reason for the difference in effect upon rich and lean mixes is obscure, but may result largely from the greater improvement in workability which a given amount of air produces in the leaner mixes. The explanation of this might be found in the viscosity and other rheological properties of the water-cement-air mixture, but few data

on this aspect are available. Lean mixes are not normally used where strength is of great importance, but in mass-concrete work, foundations, pipe haunching, and similar conditions where lean mixes and perhaps large aggregate are used, air-entrainment may be desirable in order to facilitate placing without segregation or bleeding, and the quality of the concrete will at any rate not be impaired for any given workability.

In these tests, the grading of the aggregate was not varied but, as already mentioned, the entrained air appears to supplement the fine aggregate and the amount of this can therefore be reduced. This in turn enables a small reduction in water content to be made and the strength is raised. As a result, it is possible to employ air-entrained concrete without loss of strength for richer mixes than indicated above.

A further point which should not be overlooked is the reduction in density. Comparing two mixes, one of ordinary concrete and the other air-entrained, which have the same workability and strength, the air-entrained concrete will contain 5 per cent of air and therefore 5 per cent less of the solid materials in a given volume. This will result in an economy of about 5 per cent in the cost of cement and aggregate, against which must be offset the very small cost of the air-entraining agent and the additional labour involved in dispensing it. Alternatively, the cement content may be increased without raising the cost above that of ordinary concrete, and thereby obtaining a further increase in strength. Results have recently been published [11] of tests carried out by an independent laboratory for a supplier of an air-entraining agent in Great Britain, and these show an increase in compressive strength at 28 days of about 9 per cent for 1 : 6 mixes when full advantage has been taken of all the effects of the entrained air.

Air-Entraining Materials and the Methods of using Them

Air-entrained concrete is generally produced by using an admixture, such materials being termed air-entraining agents. These agents are used in very small quantities, generally between 0·005 and 0·05 of 1 per cent of the weight of cement. It is frequently more convenient to use a larger quantity of a dilute solution.

Many organic materials have the effect of entraining air in concrete, and they may be grouped as follows :—

(1) Natural wood resins.
(2) Animal and vegetable fats, and oils such as tallow and olive oil, and their fatty acids such as stearic and oleic acids.
(3) Various wetting agents such as the alkali salts of sulphated and sulphonated organic compounds.
(4) Water-soluble soaps of resin acids, and animal and vegetable fatty acids.
(5) Miscellaneous materials such as the sodium salts of petroleum sulphonic acids, hydrogen peroxide, aluminium powder, etc.

In the United States, a number of proprietary admixtures have been developed as air-entraining agents and the two most widely used ones are available in Great Britain. The first of these is a resin and is a product of the distillation of pine stumps. It belongs to the first of the groups mentioned above and when used in this form it reacts with the free lime in the cement to form a water-soluble resinate. It is more usual to pre-neutralize the material with sodium hydroxide, and it is available in a pre-neutralized form. It then forms a member of the fourth group above. The second widely used material is stated to consist substantially of a triethanolamine salt of a sulphonated hydrocarbon, and it belongs to the third group.

In addition to these materials a number of British wetting agents, at present used widely as detergents, can be used as air-entraining agents. They are not publicized as such, however, as they have other fields of use. In addition, certain other British admixtures entrain air although they are marketed for their waterproofing or other properties.

It is generally preferred to add the air-entraining agent to the concrete in the mixing water, but in the United States of America it is sometimes interground with the cement, the product being marketed as air-entraining cement. This procedure has been adopted by one British cement manufacturer, the product being marketed as a " masonry cement." It is easier to use an air-entraining cement than a separate admixture, and its use reduces the possibility of errors on the part of the man responsible for loading the mixer. The air-entraining cement suffers from the disadvantage, however, that the amount of air entrained cannot easily be adjusted, and since such factors as type and grading of aggregate, mix proportions, and temperature affect the degree of air-entrainment, it is necessary that such adjustment can be made. If too much air is being entrained, part of the air-entraining cement must be replaced by ordinary cement. If too little air is being entrained, some additional air-entraining agent must be introduced.

Where an air-entraining agent is added to the mix as a separate ingredient, there is no difficulty in effecting the adjustments required in the amount of admixtures, and in the United States of America the human element has been largely eliminated by the use of automatic dispensing devices. A typical device consists of a small piston-pump which injects the required amount of solution into the mixer drum each time the skip is raised, a simple adjustment being provided to vary the volume of liquid injected.

Factors Affecting the Amount of Air Entrained

The use of air-entrained concrete is complicated by the fact that the amount of air entrained in a mix is affected by many factors. The principal ones are :

(1) The type and amount of admixture used.
(2) The consistence of the mix.
(3) The mix proportions.
(4) The type and grading of the aggregate.
(5) The characteristics of the cement.
(6) The length of time for which the concrete is mixed.
(7) The temperature.

Different air-entraining agents may require to be used in quite different amounts in order to entrain a given percentage of air. This is not serious, however, and in any case the amounts used are small and the cost is low. It is impossible to give general rules for the amount of admixtures required.

More air is entrained by a wet mix than by a dry one, but the complete range of mixes used in constructional work are suitable for air-entrainment. In recent investigations at the Road Research Laboratory "no-slump" concrete was used and this entrained air quite satisfactorily, while most of the American work has been carried out with concrete having slumps of 2 to 4 inches.

American tests indicate that concrete made with angular aggregates entrains a greater percentage of air than that made with rounded gravels, and also that the aggregate grading has a significant effect.[10, 12] Walker and Bloem [10] carried out tests with various gradings and observed a marked increase in air content when the percentage of material passing a No. 30 A.S.T.M. sieve and retained on a No. 50 A.S.T.M. sieve (corresponding to the fraction between the No. 25 and No. 52 B.S. sieve sizes) was raised.

The amount of air entrained varies considerably with the brand of cement and with different batches of the same brand. The reason for this is not yet clear, and the behaviour of a particular batch is not predictable.

The duration and character of the mixing both affect the amount of air entrained, but not in accordance with simple rules. It appears that the amount of air entrained increases in the first minute or two and then remains constant or may decrease slightly.[10, 13]

Walker and Bloem [10] have shown that the higher the temperature the lower is the amount of air entrained. At 50° F. the amount of air entrained is 4/3 times the amount entrained at 70° F., whilst at 90° F. the amount is only three-quarters of that entrained at 70° F.

The Measurement of Air Content

If the amount of air entrained by a mix differs widely from the design figure, the properties of the concrete may be seriously affected; too little air will result in insufficient workability, while too much air will result in low strength. It is therefore necessary that the air content should be maintained within certain limits (for example $4\frac{1}{2}$ per cent $\pm 1\frac{1}{2}$ per cent, or 5 per cent ± 1 per cent), and in view of the many factors which affect

this it must be measured frequently throughout the progress of the work. The air content may then be adjusted by altering the amount of air-entraining agent entering the mixer.

Three methods of measuring the air content of fresh concrete have been developed, and each has its own merits. They may be termed (1) the gravimetric method, (2) the displacement or volumetric method, and (3) the pressure method; the last two methods have various modifications.

The gravimetric method was the first to be used, and it did not require any special equipment. The test is principally one of determining the density of the fresh concrete compacted in a standard manner; this is then compared with the theoretical density of air-free concrete, calculated from the specific gravities of the constituents and the mix proportions. Thus if the air-free density is 150 lb. per cubic foot and the measured density is 142·5 lb. per cubic foot, then 1 cubic foot of concrete will contain 142·5/150 cubic feet of solid and liquid matter, the rest being air. The air content then is:

$$1 - \frac{142\cdot5}{150} = 5 \text{ per cent}$$

The gravimetric method has been standardized in America, where a ½-cubic-foot cylinder (with an internal diameter of 10 inches and internal height of 11 inches) is used to determine the density (provided that the aggregate is not larger than 2 inches). The concrete is compacted with a steel rod of ⅝ inch diameter as used in the slump tests. At the Road Research Laboratory, using ¾-inch aggregate, it has been found convenient to use the cylinder of a compacting factor apparatus, which has a volume of about ⅕ cubic foot (internal diameter of 6 inches, and internal height of 12 inches). Since the mixes have been rather stiff, compaction has always been carried out by vibration. It has not so far been possible to standardize the vibration, and in view of the tendency for the entrained air to be expelled, different operators might obtain slightly different results.

The gravimetric method is satisfactory for use in the laboratory, but is not well suited for field work. It necessitates a skilled operator and an accurate balance. It also requires a knowledge of the mix proportions of the concrete, including the water content, and even if all the materials are weighed carefully, this is no guarantee of the mix proportions of the concrete discharged from a mixer on the site, owing to adhesion of mortar to the mixer drum, splashing and similar factors. Further, the mix proportions of a sample taken on the site may differ considerably from the value of the mix as a whole if there is any tendency for segregation to take place. The specific gravities of the materials must also be known with considerable accuracy, and variations in these can introduce a further error into the result.

The displacement or volumetric method aims at measuring directly the volume of air in the sample. There are a number of modifications but the principle is to take a sample of concrete of known volume, remove the air and then determine the amount of water (by volume or weight) required to restore the sample to the original volume.

A vessel is partly filled with concrete, the amount being determined by weighing, or by using a vessel made in two parts, the lower one being filled completely and struck off level so that a fixed volume of concrete is obtained. Water is added to make up a given volume as indicated by a mark on a narrow neck, a hook gauge or other device. The air is then removed by agitation of the mixture (either by rolling on the bench or in a cradle, or by stirring) and when the removal is complete the water level is restored to its original position by a further addition. The additional water may be measured from a measuring cylinder or burette, or the amount may be determined by noting the increase in weight of the vessel and its contents.

In some variations of this method, weighing is eliminated entirely. The rest of the test requires great care, however, and a skilled operator is required. If the added water is measured by volume, care must be taken to avoid any loss of water by splashing or evaporation during the removal of the air, while if it is determined by the increase in weight, an accurate balance and careful weighing are necessary. It is in any case a tedious job to remove all the air, and it is difficult to know when the removal is complete. The trouble caused by the formation of a scum on the surface is often reduced by using for the final addition a water-alcohol mixture or other liquid which breaks up the bubbles.

The pressure method has become very popular in America and is probably the best for field use. It is based on the decrease in the volume of air when a pressure is applied. In the type of instrument shown in *Fig. 8* (facing p. 344) and *Figs 9* a vessel is filled with concrete compacted in a standard manner, and struck off level. A cover is then clamped in position, this being in the form of an inverted funnel, consisting of a metal cone, and a glass gauge-tube provided with a scale. Water is added until the level reaches a zero mark on the tube and then a pressure is applied by means of a bicycle pump. The pressure is transmitted to the air entrained in the concrete, which contracts accordingly, so that the water level falls. The pressure is increased to a pre-determined value as indicated by a small pressure-gauge, and the glass gauge-tube is so calibrated that the percentage of air by volume is indicated directly. The working pressure to be used must be determined beforehand, and is such that the graduations on the gauge tube represent the air content to a convenient scale (frequently 1 inch = 1 per cent air content). The instruments are generally designed to employ a working pressure of the order of 15 lb. per square inch. A correction has to be made for the air contained in the aggregate, and can be determined easily by carrying out a test on the aggregate alone.

The pressure method is the simplest method to carry out and does not require a skilled operator. The apparatus is compact, robust, and self-contained. No knowledge of the mix proportions or densities of materials is required, and no calculation is involved. In the United States of America a number of commercial " air-meters " of this form are available and one type is available in Great Britain. This method may not be suitable for mixes of very low workability, but in any case air-entrainment is generally associated with mixes of medium and higher workability. The method has been used in America with mixes having slumps as low

Figs 9

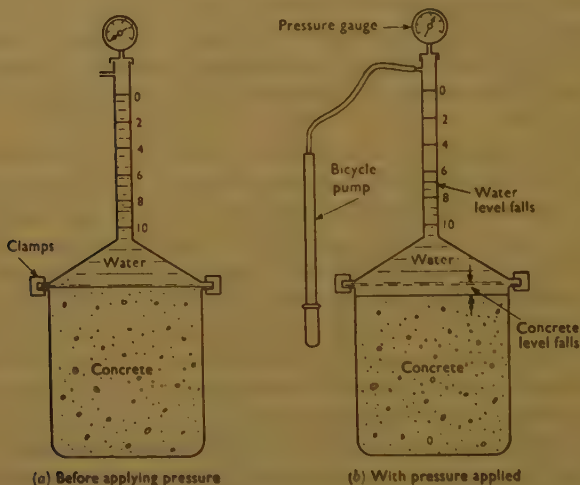


DIAGRAM OF PRESSURE-TYPE APPARATUS FOR DETERMINING THE AIR CONTENT OF AIR-ENTRAINED CONCRETE

When the air pressure is increased, the air in the concrete is reduced in volume ; this slightly lowers the surface of the concrete and causes the level of the water in the tube to fall)

as $\frac{3}{4}$ inch [14] without calling forth comment on the suitability of the method with dry mixes.

The relative accuracies of the three methods have been discussed at length by others [15, 16] and some comparative tests using the gravimetric method and three different pressure-type air-meters have been made at the Road Research Laboratory.[20] There appears to be no significant difference between them on the score of accuracy and reproducibility of results. The choice of method must therefore be made on the basis of convenience, simplicity, speed, and availability of apparatus ; whilst the gravimetric method is suited to use in the laboratory, the pressure method will generally be considered most satisfactory in the field.

Mix Design for Air-Entrained Concrete

The entrainment of air necessarily adds another factor to be taken into account when designing a concrete mix, and methods have been suggested [17, 18] for designing mixes for air-entrained concrete. It is considered that the details of a mix, whether for ordinary or air-entrained concrete, can only be finally established after trial batches have been made. The first concrete to be produced on any job should always be considered as being somewhat experimental and provision should be made to modify the mix if necessary as the experience of the engineer dictates. A final design for a mix cannot be produced on paper owing to the variations in the properties of the materials available, and therefore what is required is a simple method of estimating proportions suitable for the trial batches. The method outlined below consists in designing a mix suitable for ordinary concrete and modifying this to allow for the entrained air, while maintaining the same cement content per cubic yard of concrete and the same workability. The procedure is best carried out in tabular form, and an example is shown in Table 1. The method is as follows :—

Stage

(1) The mix is designed as for normal concrete, by any reliable method, such as that set out in Road Note No. 4.[19] The proportions by weight are written down, including the water/cement ratio, for example, 1 : 2½ : 5/0·75.

(2) These weights are converted to *absolute volumes*, by dividing by the *specific gravities* of the individual constituents.

(3) These volumes are expressed as a percentage of the whole.

(4) Assuming " *A* " per cent of air is to be incorporated, " *A* " is subtracted from the percentage of aggregate and " *A* " per cent of air is added (this will require a new column) so that the total remains 100. (In Table 1, " *A* " = 5.) Since the air apparently behaves as particles of fine aggregate, fine aggregate alone might be subtracted to allow for the air. This, however, would tend to produce a harsh mix and therefore the majority of the aggregate removed should be fine and a smaller amount (experience suggests about 1 per cent) should be coarse.

(5) The added air will have increased the workability and the increase must be counteracted by reducing the water content. For each 1 per cent of air added, " *W* " per cent of water is removed, where " *W* " is taken from Table 2, and this is replaced by an equal volume of aggregate. The proportion of coarse to fine aggregate added should be approximately the same as the existing proportions ; it is immaterial whether the proportions used for this adjustment are the original ones or the modified ones, as the difference would be negligible.

The figures in the line thus obtained represent the percentage absolute volumes in the air-entrained mix.

(6) These volumes are converted to weights by multiplying by the specific gravities of the constituents.

(7) Each weight is divided by the weight of cement to obtain the mix proportions and water/cement ratio in the usual form.

A number of points should be noted in connexion with this method of designing an air-entrained mix. It must be appreciated that the volume ratios obtained in Stage 2 are not the mix proportions by volume. These are in absolute volumes, whereas the mix proportions when given by volume are in terms of bulk volumes. If the mix is initially designed by volume, the proportions should immediately be converted to proportions by weight by multiplying by the bulk densities.

Stage 2 requires the knowledge of the specific gravities of the constituents but they are not required to a high degree of accuracy, since the process is reversed in Stage 6. Wide variations have only a small effect on the results of these calculations and, if no other information is available, a value of 2·6 or 2·7 can be used for most concrete aggregates. The specific gravity of Portland cement may be taken as 3·12, and its reciprocal as 0·32.

The values of "*W*" required in Stage 5 were obtained from a comparatively small number of tests and should not be considered to be final. Further work is needed to supplement these data.

TABLE 1.—TYPICAL MODIFICATION OF MIX DESIGN FOR AIR-ENTRAINED CONCRETE USING ¾-INCH IRREGULAR GRAVEL AGGREGATE

Stages of procedure	Cement	Fine aggregate	Coarse aggregate	Water	Air	Total
(1) Mix proportions by weight	1	2¼	5	0·75		
(2) Divide by specific gravities	$\frac{1}{3\cdot12}$	$\frac{2\frac{1}{4}}{2\cdot68}$	$\frac{5}{2\cdot58}$	$\frac{0\cdot75}{1}$		
Absolute volumes . .	0·32	0·93	1·94	0·75		3·94
(3) Per cent absolute volumes .	8·1	23·6	49·3	19·0		100·0
(4) Add air and subtract aggregate	—	4·0	1·0	—	5·0	
	8·1	19·6	48·3	19·0	5·0	100·0
(5) Subtract water and add aggregate	—	0·7	1·5	2·2	—	
	8·1	20·3	49·8	16·8	5·0	100·0
(6) Convert to weights . . .	25·3	54·4	128·5	16·8		
(7) Divide by weight of cement	1	2·15	5·08	0·664		

TABLE 2.—VALUES OF "W" FOR $\frac{3}{4}$-INCH AGGREGATE

Mix proportions	Rounded gravel aggregate	Irregular gravel aggregate	Crushed rock aggregate
1 : 6	0·325	0·375	0·425
1 : 7½	0·40	0·45	0·50
1 : 9	0·45	0·50	0·55

The amount of air-entraining agent necessary to produce the required air content must be found by trial and error. It cannot be estimated accurately in view of the many factors affecting the air content.

From the example of mix design quoted in Table 1, the reduction in strength owing to the entrainment of air can be estimated. The original mix had a water/cement ratio of 0·75, which corresponds to a compressive strength of 2,700 lb. per square inch at 28 days. The modified mix has a water/cement ratio of 0·66 which would correspond to a strength of 3,500 lb. per square inch for air-free concrete. Since each one per cent of air reduces the strength by 5·5 per cent, the reduction in this case would be $5 \times 5{\cdot}5 \times 3{,}500/100$ lb. per square inch, that is, 1,000 lb. per square inch, and the strength would be $3{,}500 - 1{,}000 = 2{,}500$ lb. per square inch; this represents a reduction of 200 lb. per square inch compared with normal concrete. If an average strength of 2,700 lb. per square inch had been required it would be necessary to re-design the initial ordinary mix to give a strength of 2,900 lb. per square inch, and carry out the modifications again.

ACKNOWLEDGEMENTS

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The Paper is published by permission of the Director of Road Research.

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Note.—References Nos 1, 4, 12, and other Papers are issued as a reprint entitled "Air entrainment in concrete" (Book 1).

References Nos 3, 5, 9, 10, 13, and other Papers are issued as a reprint entitled "Air entrainment in concrete" (Book 2).

References Nos 14, 15, 16, and other Papers are issued as a reprint entitled "Symposium on measurement of entrained air in concrete."

The Paper is accompanied by two photographs and seven sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

CORRESPONDENCE

on a Paper published in Proceedings, Part I, March 1953

Paper No. 5882

"Creep of High-Tensile Steel Wire" †

by

Noel William Bailey Clarke, M.Eng., M.I.C.E., and
Francis Walley, M.Sc., A.M.I.C.E.

Correspondence

Mr A. J. Harris, referring to the Authors' mention of the high creep values found by Simon and Xercavins, observed that those values were employed in practice by S.T.U.P. ("Société Technique pour l'Utilisation de la Précontrainte") in estimating stress relaxation when using the type of French steel in question.

There was a reason for supposing that creep values were closely connected with the degree of homogeneity of the crystalline structure and internal stress of the steel. According to that theory, creep consisted of plastic strains being reached in limited zones of the wire; those zones transferred stress to other zones which were still elastic and a slight extra elongation resulted. Thus the greater the homogeneity and the smaller the "locked-up" stresses in the unloaded state, the smaller the creep; the high values found by Simon and Xercavins were probably to be explained by features in the manufacturing process—for example, too small a number of passes through the dies.

It should be remarked that a new type of process developed in France especially for prestressing wire, and which combined cold-drawing with heat-treatment, appeared to give lower results than those quoted by the Authors, though more work was needed to give conclusive results.

Mr J. L. Bannister, observed that the Authors had claimed to have investigated two variables, namely, diameter and ultimate strength. The characteristics of drawn wire were not a simple function of either diameter or ultimate strength, but were dependent upon the basic material and its treatment, and the extent and manner of subsequent cold reduction. The resultant properties, as the Authors had pointed out, were "a function of

† Proc. Instn Civ. Engrs, Part I, vol. 2, p. 107 (March 1953).

the amount of drawing which takes place, and any work- or age-hardening which may occur." A variation in diameter of a drawn wire inferred both a variation in the diameter of the basic material prior to drawing, and a variation in the rapidity of the total reduction in cross-sectional area.

The condition of the 0·2-inch-diameter wire in " small " and " large " coils immediately prior to testing was not made clear, and before making any comparison between the creep losses on the basis of coil diameter, it was essential to know that condition. The stress/strain curves shown in *Fig. 7* indicated that both the " small " and " large " coil wires exhibited elastic characteristics in the initial stages of stressing, which suggested that both had received some form of treatment subsequent to cold-drawing. Otherwise they would exhibit the plastic characteristics of wire in the as-drawn condition.

It was well known that drawn wire would strain-age at ordinary temperatures. By artificially strain-ageing the drawn wire at higher temperatures, the behaviour of the wire under tensile stresses was more elastic and stable, but the creep losses were increased. It had also been demonstrated, by Magnel, Strycker, and others, that by stressing the wire in tension to a value higher than the service stress for a short time, the creep losses were decreased. Thus it was clear that the behaviour of the wire with stress and time was dependent upon treatment subsequent to drawing, and if the " small " coil had been strain-aged and the " large " coil had been previously tensioned, that would explain the difference in behaviour of the two coils.

Would the Authors say whether the coils used in their experiments had received such treatment subsequent to drawing ? If so, what precisely had been the nature of such treatment ?

Whilst from the Authors' tests it was apparent that the 0·2-inch-diameter " large " coil had smaller creep losses, it could be seen in *Fig. 7* that for the same concrete deformation the loss of steel stress from 200,000 lb. per square inch was greater for the " large " coil than for the " small " coil.

Mr W. O. Everling, of Cleveland, Ohio, observed that the apparatus used by the Authors in making their tests was ingeniously contrived.

The research laboratory of the American Steel and Wire Division of the United States Steel Corporation had been making similar tests on a 100-foot-long test-bed constructed about 8 months previously. The wire was suitably supported at frequent intervals and the progression of stress relaxation tested with a specially designed load cell. Needless to add, those researches were by no means complete.

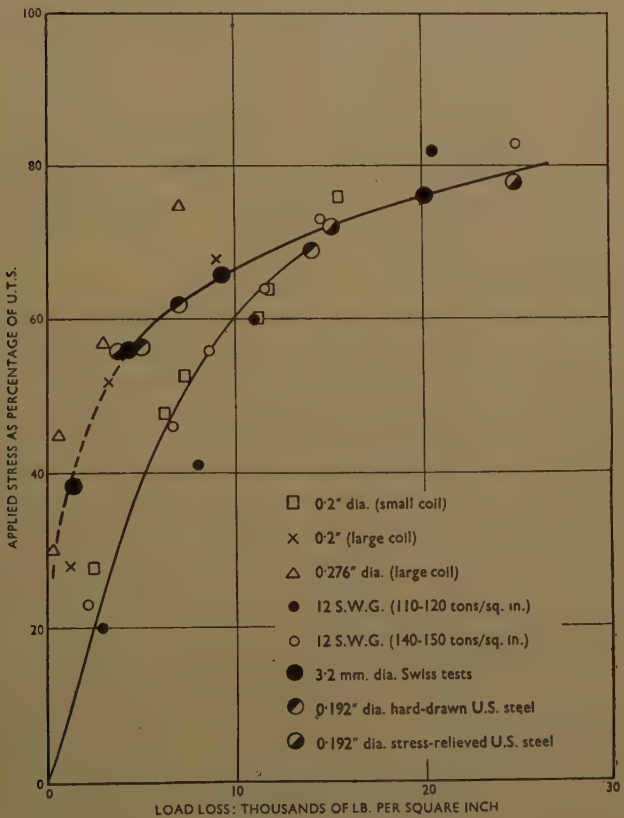
A comparison of results with those of the Authors was the more interesting because they agreed so well with a part of them, and so poorly with another part.

The most popular wire used for prestressing in the United States had 0·75 to 0·80 carbon, medium manganese, and was cold-drawn to a tensile

strength of about 250,000 lb. per square inch. The most common diameter was 0·192 inch, both for single wires and for heavy strands (larger than 0·600 inch diameter).

In the state in which such wire came off the final wire-drawing block it had a free coil set of about 36 inches. Its stress/strain relationship was typically soft; that was to say, it yielded plastically and quite measurably at stresses as low as 60,000 to 80,000 lb. per square inch.

Fig. 25



During recent years it had become popular in the United States to give such wire a final "time-temperature" or "stress-relieving" treatment, which increased the original elongation of about 2½ per cent to the minimum of 4 per cent measured in a gauge length of 10 inches. It also affected the stress/strain curve, making the wire substantially elastic up to at least 50 per cent, and perhaps slightly more of its ultimate strength.

Incidentally, that treatment did not change the original strength significantly, while at the same time it straightened the wire itself quite well. The final curvature was approximately one foot in a length of 12 feet.

That relative straightness had become an important consideration because it facilitated handling in the field and the higher apparent elastic limit had suggested that it might have higher creep resistance, particularly at very high stress.

The U.S. test programme was designed to compare the resistance against stress relaxation of the wire in the hard-drawn state with the same type of wire in the stress-relieved state. In the light of the Authors' programme, it also afforded an opportunity to compare wire with a free coil set of only 36 inches with almost straight wire.

Fig. 25 showed the results of the American investigation. Somewhat unexpectedly, both the stress-relieved wire as well as the wire in the as-drawn state, showed about the same degree of stress relaxation between the limits tested, namely 55 to 75 per cent of ultimate strength. It was also remarkable to see the close agreement with the Authors' 0·2-inch (large coil) and the Swiss 3·2-millimetre wire, as well as a part of the Authors' 0·276-inch wire.

On the other hand, the American researchers found it difficult to explain the rather wide discrepancy between their results and those obtained from the Authors' small-coil wires. It would be interesting to have more details of those wires regarding analysis and elongation, particularly in comparison with the large-coil wires. One effective way of determining elongation was to measure what was called in America the uniform elongation. That was done by marking a gauge length of perhaps 20 inches at intervals of 1 inch. The sample was then broken under tension and the percentage elongation determined on a piece of the wire which did not include the necked-down zone of the actual break.

It had also occurred to Mr Everling and his associates that the handling of the small-coil wire might have presented considerable difficulties A certain amount of hand straightening might have had to be done. The possibly resulting faint " kinks " might have affected the relaxation results considerably, particularly in the lower stress-range.

Mr Everling hoped that it would be possible to continue to compare results as time progressed.

Mr S. C. C. Bate observed that reference was made in the Paper to the investigation of the properties of hard-drawn wire for prestressed concrete, which was being carried out at the Building Research Station. That investigation included a number of relaxation tests on different samples of wire of British manufacture, in which some of the specimens had been under load for more than 2 years. The results for relaxation at 1,000 hours were of the same order as those obtained by the Authors, although the difference between the behaviour of the straight wire from the large-diameter coils and the unstraightened wire from small-diameter coils was

not so distinct. The results for relaxation after 10,000 hours gave some indication of the errors introduced by extrapolation from the results of tests of short duration. For the wires from small-diameter coils, the errors through extrapolation appeared to be of little practical importance, and an approximate value for the relaxation during that period might be obtained from tests of much shorter duration than 1,000 hours. The errors involved in extrapolation were, however, greater for the relaxation of the particular samples of straight wire from large-diameter coils, which were being tested at the Building Research Station. It had been found that, as the duration of the tests increased, the difference between the relaxation of the wires from the coils of small diameter and of those from the coils of large diameter became less marked for initial stresses between 60 per cent and 70 per cent of the tensile strength and negligible at higher initial stresses. The range of values for relaxation over a period of 10,000 hours for three different samples of wire from small coils and two different samples of straight wire from large coils was $3\frac{1}{2}$ to 7 tons per square inch, for an initial stress of 60 per cent of the tensile strength, and 6 to $9\frac{1}{2}$ tons per square inch for an initial stress of 70 per cent of the tensile strength.

The Authors had suggested that the effects of relaxation could best be offset in practice by permanently overstressing the wires as an alternative to the slower process of temporarily overstressing for a few minutes before anchoring. The results of several tests made at the Building Research Station on the effect of temporarily overstressing the wires had shown that the subsequent reduction in relaxation was the result of a reduction in the rate of relaxation during the first few days only and thereafter the rate was unaffected. As a result of temporarily over-stressing by 10 per cent for 1 minute, the relaxation of a sample of wire in 1,000 hours was reduced from 5·3 tons per square inch to 3·6 tons per square inch for an initial stress of 70 per cent of its tensile strength. The reduction in relaxation was therefore only a small proportion of the applied stress and could be more conveniently offset in practice by a permanent increase in the initial stress, as suggested by the Authors.

During recent years the creep and relaxation of steels for prestressed concrete had attracted considerable attention, and since the Authors had started their tests more published information had become available. Strict comparisons between the results of different investigations could only be made if details of the experimental technique were given; the rate of stressing and the method of establishing the initial condition for the test might have a considerable influence on the results obtained subsequently. It would, therefore, appear desirable to give the details of the stress- and strain-history of the material in presenting the results of creep or relaxation tests.

The Authors, in reply, stated that they were glad that the figures quoted in the opening remarks to the Paper concerning the French figures

for relaxation were not generally applicable. In their view it was too early to say whether the new type of steel, referred to by Mr Harris, would have superior properties to the present wire used in Great Britain.

In reply to Mr Bannister, the Authors would agree that the two variables referred to were meaningless in themselves, since both sizes and strengths of wire would not be drawn from the same original wire. "Investigated" was too strong a word; they had used the wires described to cover a range of ultimate tensile strength and diameter. The condition of the 0·2- and 0·276-inch-diameter wire in the large coils had now been made clear by Mr Brereton's contribution; the condition in the small coils was, so far as the Authors were aware, in the as-drawn state. They were not aware that relaxation losses were increased by strain-ageing at higher temperatures since that was the treatment of the wire on the large coils which showed a marked decrease in relaxation. The wire in the large coils had not been previously tensioned. The Authors were not clear about the reference to *Fig. 7*. It seemed to them merely to show that the strain at a stress of 200,000 lb. per square inch in the wire from the large coils was smaller than for the wire in the small coils.

They were interested to learn from Mr Everling's contribution of the work of the American Steel and Wire Division of the United States Steel Corporation. They were surprised to see that the same relaxation was obtained with stress-relieved wire as with wire in the as-drawn state, but were pleased to see that their results were of the same order of magnitude as those described in the Paper. They looked forward to seeing further results from the American tests in due course.

They were grateful to Mr Bate for his contribution, and only wished that it could have been amplified, because it was clear that he had much valuable data. They were surprised to learn that the rate of stress loss for the wire from large coils did not decrease as quickly as they had estimated, and would be glad to know what the difference between the relaxation of the two types of wire was at 10,000 hours for an initial stress of 60–70 per cent of their ultimate strength. It was encouraging to learn that his tests confirmed that the best method of taking care of the relaxation was permanently to overstress the material initially. They agreed with Mr Bate as to the necessity of providing full data when presenting the results of similar experiments; reconciling experimental data often proved impossible because of lack of data.

A general point which emerged from the discussion which should be emphasized was the necessity for the designer in prestressed concrete to have a knowledge of the treatment to which the wire he proposed to use had undergone. When that information was obtained it should be possible for him to know into which category he should place the wire and so have an idea of the possible relaxation which could take place.

In conclusion, the Authors would like to thank all the contributors to

the discussion. They felt that the interest which had been aroused had justified the presentation of their Paper, and hoped that it would stimulate others to carry out further work on a relatively new subject so that a definite result may be arrived at which would be of direct service to designers in prestressed concrete.

OBITUARY

SIR DAVID ANDERSON, who died at Cupar, Fife on the 27th March, aged 72, was born on the 6th July, 1880 at Leven, Fife. His general education was at Dundee High School. Later he studied civil engineering at St Andrews University and obtained his degree (B.Sc.) there in 1900.

After that he served a period of practical training, first at the Glasgow works of Sir William Arrol and Company, Limited, the famous bridge builders and general steelwork firm. Later, in 1905, he came to London and served under Sir Benjamin Baker, K.C.B., the designer of the Forth Railway Bridge and many other large and important engineering works.

Sir Benjamin Baker had delegated the construction work on the widening of Blackfriars Bridge to Mr Basil (later Sir Basil) Mott, and Mr Anderson was appointed Resident Engineer for that widening work in 1906. Sir Benjamin died early in 1907. Mr Mott had been associated for some years with Sir Benjamin on the design of the City and South London tube railway and Mr Anderson took part in many later works on these railways. He was also engaged on several important bridge works at about this period, notably the new Southwark Bridge across the Thames and at Rochester Bridge across the Medway. He was chief assistant to Mr Mott.

In the First World War, Mr Anderson served in the Royal Engineers and completed his military service as a captain.

In 1923 he became a partner with Mr Basil Mott and Mr David Hay, who had been partners for a considerable time before that and had been responsible for many important bridge and tunnel works.

Following the end of the war, the firm of Mott, Hay & Anderson entered into a period of great activity, and Mr Anderson, in collaboration with his partners, was engaged on the design and supervision of construction of numerous large and important bridge and tunnel works. One of the tunnel works was the Mersey Tunnel between Liverpool and Birkenhead. This was the largest sub-aqueous vehicular tunnel in the world. This work was completed, and another important tunnel work was that for the Dartford-Purfleet tunnel under the Thames. This work was started and the first part of it—the pilot tunnel—was completed, but the construction of the full-size vehicular tunnel was deferred and it has not yet been started. Mr Anderson's firm was associated in this tunnel work with Messrs Coode and Partners. There were also many miles of London underground railway tunnels, and numerous escalator tunnels in conjunction with them. Among the bridges were the Tyne Bridge at Newcastle, Wearmouth Bridge, and the Tees (Newport) Vertical Lift Bridge.

In addition to works which went into construction, Mr Anderson and

his partners studied and reported on many important works, some of them of considerable magnitude: the proposed Channel Tunnel was one of the latter. In addition, he and his firm were actively engaged on the design for the proposed long-span suspension bridges across the Rivers Forth and Severn. Mr Anderson (as he still was) was also responsible for the design of the pedestrian tunnel and the vehicular tunnel under the River Tyne.

He served on many important technical committees for the Government, and was Chairman of some of them.

He was created a Knight Bachelor in June 1951.

He was elected an Associate Member of the Institution in 1906, and was transferred to full membership in 1915. In 1930 he was elected to the Council and was President during Session 1943–44. He was Author of several Papers which were read before the Institution. He was also a member of the American Society of Civil Engineers.

He was married first to Isabella Corbet Anderson, daughter of Mr Robert Anderson of Dundee; she died in 1929, and in 1935 he married Agnes Gilchrist Anderson, who survives him.

JAMES CROSS, who died at London on the 27th January, 1953, at the age of 47, was born on the 7th August, 1905.

He was educated at Sherborne School, and at Loughborough College, Leicester, where he was awarded an honours diploma in engineering.

On leaving College in 1926, he joined Sir Douglas Fox and Partners as Assistant Engineer during the construction of the hydro-electric power scheme at Maentwrog, North Wales.

In 1927, he left England to take up an appointment in South America, with Messrs Adrian Heriot and Company of Buenos Aires, and for the next five years was engaged on various works of construction for the Buenos Aires and General Railway, and also for the Central Argentine Railway.

He returned to England in 1932, joining Messrs Wilson Lovatt and Sons as Engineer, and was engaged on the construction of roads and sewers in the London area.

In 1935, however, Mr Cross left England again to be Agent to Messrs Pauling and Company on the construction works in the Argentine and, in addition to the building of many roads, he was responsible for the construction of the San Felipe dam.

From South America he went, in 1941, to the Sudan, where he was appointed Harbour Engineer to Port Sudan; whilst in this post he was instrumental in the building of new quays, railway sidings, and a new reservoir.

Returning to England in 1943, he joined the staff of the Ministry of War Transport, being attached to the Port and Transit Section as Deputy Harbour Engineer.

In 1946 he was appointed Harbour Engineer to the Colombo Port Commission, Ceylon, a post which he held until 1952, when he joined the

staff of Binnie, Deacon and Gourley for whom, but for his death, he would shortly have gone to Iraq as Resident Engineer on a high dam.

Mr Cross was elected an Associate Member of the Institution in 1939 and was transferred to the class of Member in 1945.

In 1941 he was awarded a " Follett Holt " premium for his Paper " San Felipe Dam Construction " read before the Buenos Aires Local Association of the Institution.

He was a Member of Council (Ceylon) from 1950 to 1952.

He is survived by his widow.

ALFRED CHARLES GARDNER, F.R.S.E., who died on the 2nd December, 1952, at the age of 72, was born at Richmond, Surrey on the 4th October, 1880.

He was educated at the Vineyard School, Richmond, Surrey, and at the Polytechnic School of Engineering, London.

He obtained his practical experience by serving an apprenticeship with the Thames Ironworks, Shipbuilding, and Engineering Company, of Blackwall, London, from 1898 to 1901.

On the completion of his training, he was appointed Assistant in constructional steelwork in the Highways Department of the London County Council, and was engaged in the design of the London County Council Generating Station at Greenwich.

He left this post in 1901 to become an Assistant in the Engineering Department of the Great Western Railway, where he was concerned with with the design of bridge and roof work.

He left this latter railway company in 1907 to join the staff of another —the Great Central Railway Company—first from 1907 to 1909, as Steelwork Assistant, and later as Principal Bridge Assistant in the New Works Department, and in the years that followed he was responsible for the renewal and construction of over thirty bridges. The most important of these was the new bridge over the River Trent at Keadby, which, at the time, was the largest Lifting Bridge in Europe.

This latter period of his career was terminated by his joining the London and North Eastern Railway Company in 1918, as Docks Engineer, and as such he was subsequently responsible for the design and construction of Grimsby and Immingham Docks, and also the Lifting Bridge across the Alexandra Dock, Grimsby, which was opened in July 1928.

In the following year he was appointed Chief Engineer to the Clyde Navigation Trust, where he was responsible for the Docks and Harbour of Glasgow, and the maintenance of the Clyde. He retired from the post in 1941. The major works undertaken during this period included the reconstruction of General Terminus Quay, Plantation Quay, and Lancefield Quay ; the foundations for the 175-ton crane at Stobcross Quay ; and the various river works in connexion with the launching of Q.S.T.S. Queen Mary and Queen Elizabeth, involving the deepening of the River Clyde over

a length of 14 miles, the widening of the River at two points, and deep dredging on the line of the launch.

Mr Gardner was elected an Associate Member of the Institution in 1909, and transferred to the class of Member in 1918. He was Chairman of the Glasgow & West of Scotland Association in 1946–47 and was elected a Member of the Council in 1952.

In 1934 he was awarded a Telford Premium for his Paper " Construction of No. 2 Graving Dock at Elderslie," [1] and he also delivered the Vernon-Harcourt Lecture for 1939-40 on " The Construction of Deep-Water Quays." [2]

Mr Gardner was a Fellow of the Royal Society of Edinburgh, a Member of the Institutions of Mechanical and Electrical Engineers, and was also a Past-President and Honorary Member of the Institution of Engineers and Shipbuilders in Scotland.

He leaves a widow and a son and daughter.

PROFESSOR ALEXANDER HOPE JAMESON, M.Sc., who died on the 23rd December, 1952, at the age of 78, was born at Highbury, London, on the 15th October, 1874.

He was educated privately and at Owens College, Manchester, which he entered as a student in 1891 and where he graduated with honours in Engineering in 1894. He remained at the College for a further three years as Bishop Berkeley Fellow and Demonstrator in the Whitmore Engineering Laboratory.

He left this post in 1897 to spend three years as a pupil in the Engineering Department of the Lancashire and Yorkshire Railway.

In 1901, at the completion of his practical training, he joined the staff of the Derwent Valley Water Board, first as an Assistant to Mr Edward Sandleman ; then from 1905 until 1909 as Resident Engineer on the construction of the Main Aqueduct from Grindleford to Rowsley, and finally from 1909 to 1912 he held a similar post during the construction of the Thirlmere Aqueduct (third pipeline).

In 1912, he was appointed to the Chair of Civil Engineering at King's College, Strand, this he held until his retirement in 1935.

Professor Jameson was Dean of the Faculty of Engineering, University of London, from 1932 until 1935.

He was also author of several standard textbooks, including " Advanced Surveying," " Fluid Mechanics," " Contour Geometry," and " Mathematical Geography."

He was awarded a James Forrest Medal and a Miller Prize in 1896 for his Students Paper on " The Strength of Materials," and also presented to

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[1] Sel. Engng Paper No. 160, Instn Civ. Engrs, 1934.
[2] J. Instn Civ. Engrs, vol. 14, p. 129 (April 1940).

the Institution a Paper on "Flow over Sharp-edged Weirs: Effect of Thickness of Crest"[1] in 1948.

Elected Associate Member in 1901, Professor Jameson was transferred to the class of Member in 1912. He was for many years a member of the Research Committee on Velocity Formulae for Open Channels and Pipes.

He leaves a widow.

BRIGADIER-GENERAL MAGNUS MOWAT, C.B.E., T.D., F.R.S.E., who died in London on the 19th January, 1953, at the age of 77, was born on the 10th November, 1875.

He was educated at Aberdeen Grammar School, and Blackheath School, and at King's College, London, where he obtained a diploma in engineering.

On leaving College in 1896 he became a pupil to Mr W. H. Holmes, Superintendent of the locomotive shops of the North British Railway at Cowlairs, Edinburgh. On the completion of his pupilage in 1898, he became Resident Engineer and Assistant to Mr E. Parry, M.I.C.E., during the construction of the London to Leicester section of the Great Central Railway.

In 1899, he went to India, first as Assistant Engineer to the Indian Midland Railway at Jhansi in the North West Province, and later as Resident Engineer at Agra for the Great Indian Peninsular Railway.

Returning to Great Britain in 1901, he joined Sir Robert McAlpine and Sons Ltd, as Resident Engineer during the construction of the Partick section of the Glasgow main drainage.

In 1902 he was appointed Chief Assistant to Mr F. E. Duckham, M.I.C.E., Engineer of the Millwall Dock Co., becoming Engineer in 1905; after the formation of the Port of London Authority in 1906 he remained in the charge of the Millwall Docks and the East and West India Docks.

During the First World War, General Mowat, who had been a Territorial officer, served with the Royal Engineers, and for 2 years he was in charge of a division. He then held many important posts including Commandant of the School of Heavy Bridging and Pontooning, and also Commands Roads Officer at the War Office. At the close of the war he was deputy chairman and administrative officer of the Joint Roads Committee.

On leaving the Army he was given the honorary rank of Brigadier-General and in 1919 he was appointed a Commander of the Order of the British Empire.

In 1920 he succeeded the late Mr E. T. Worthington as Secretary of the Institution of Mechanical Engineers, being obliged to retire in 1938 through ill-health.

General Mowat was elected an Associate Member of the Institution in

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[1] J. Instn Civ. Engrs, vol. 31, p. 38 (Nov. 1948).

1901, and was transferred to the class of Member in 1909, in which year he read a Paper on "Some Recent Grain-handling and Storing Appliances at the Millwall Dock"[1] before the Institution.

He was a Member of the Institution of Mechanical Engineers and the Institution of Engineers and Shipbuilders in Scotland, a Fellow of the Royal Society of Edinburgh, and of the American Society of Mechanical Engineers.

He was also a liveryman of the Worshipful Company of Clockmakers of the City of London.

EDWARD JOHN SILCOCK, who died in South America on the 16th January, 1953, at the age of 91, was born on the 1st January, 1862.

He was educated at Norfolk County School, and at Yorkshire College, Leeds. In 1879 he became pupil to the City Engineer of Leeds, and on the completion of his pupilage in 1881 he remained in the City Engineer's Office first as Engineering Assistant, and later a Chief Assistant.

In 1887 he was appointed Borough and Port Engineer of King's Lynn; during his tenure of this office he was responsible for the construction of a new water supply system for the town, in addition to a system of sewerage, and the relighting of the ship channel.

In 1898 he left this post to set up practice in Leeds as a Consulting Engineer. He retained his connexion with King's Lynn, however, by becoming Engineer to the Harbour Board. As a Consultant, Mr Silcock specialized in the design of waterworks and water supply systems, sewerage and sewage disposal systems, and land drainage.

Subsequently he also opened an office in Westminster and built up a considerable practice in Parliamentary work. He retired in 1939.

During the First World War he designed a water supply system and sewage disposal system for a vast military camp at Ripon.

In addition to the several Papers read before the many Institutions of which he was a member, Mr Silcock wrote "Sanitary Engineering," and was also responsible for the section on "Sewerage and Drainage" in Kempe's Engineers' Year Book.

Elected an Associate Member of the Institution in 1897, he transferred to the class of Member in 1901.

He was also a Fellow and former Vice-President of the Royal Institution of Chartered Surveyors, a Member of the Institution of Municipal Engineers, and a Fellow of the Geological Society.

In 1887 he married Miss Annie Elizabeth Hewson, eldest daughter of Thomas Hewson, M.I.C.E., by whom he had three daughters and one son. Mr and Mrs Silcock lived to celebrate their Diamond Wedding in Buenos Aires in 1947. They left twelve grandchildren, and sixteen great-grandchildren.

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[1] Min. Proc. Instn Civ. Engrs, vol. 177 (1908–09, Pt 3), p. 58.

GUTHLAC WILSON, D.Sc.(Eng.), S.M., who was killed in an air crash near Dar es Salaam on the 29th March, 1953, was born in London on May 21st, 1902, the son of the late Henry Wilson, architect, sculptor, and art metal worker. He was educated privately and at East London College (now Queen Mary College), London.

From 1921 to 1923, he served under articles as Junior Assistant Engineer with Sir Robert McAlpine and Sons. He then worked for a number of years in India, starting in the Bombay office of Braithwaite and Co. (Engineers), Ltd. As an Assistant Resident Engineer, and later as a Resident Engineer, with this firm, he was responsible for the design and construction of a number of bridges and other works.

In 1935, he was placed in charge of the Design and Estimating Department of the Braithwaite, Burn and Jessop Construction Co.

In 1938, Dr Wilson resigned from Braithwaite & Co. and went to Harvard University, where he spent a year studying soil mechanics and obtained his Master of Science degree. He was awarded a fellowship at Havard which he subsequently resigned at the outbreak of war in order to return to England. He was the outstanding student of the year and was selected by Professor Terzaghi to act as his personal assistant on consultant work that the Professor was undertaking at the time in connexion with rolled-fill earth dams.

After his return in 1939, he was employed by Sir Alexander Gibb and Partners as a Divisional Superintendent, Designs Division, and was engaged on the design of a Royal Ordnance Factory. Subsequently, in November 1940, he became Director of Constructional Design for the Ministry of Works and Buildings, and represented the Ministry on numerous Research Boards and Committees.

From 1944, he practised privately, specializing in soil mechanics and foundation design. A year later he joined the late W. L. Scott, M.I.C.E., to form the firm of Scott and Wilson, Consulting Engineers, Westminster, and became the senior partner after the death of Mr Scott in 1950.

In the course of his practice, Dr Wilson was responsible for the design of many large structures in Great Britain, for a number of interesting works in Nyasaland comprising roads, airfields, and earth dams, and for other works in Cyprus, the Middle East, and India. During the last few months of his life he had twice visited Hong Kong in connexion with the scheme for a new airport at Kai Tak. He was consulted on many difficult problems in connexion with foundations and by the time of his death had achieved international recognition as an authority on the science of foundation engineering.

Dr Wilson was elected an Associate Member of the Institution in 1928, transferred to the class of Member in 1938, and became a Member of Council in 1949.

He was awarded a Telford Premium in 1942 for his Paper on " Calcula-

tion of the Bearing Capacity of Footings on Clay," [1] and another, in 1943, for a Paper written in collaboration with Mr Henry Grace, A.M.I.C.E., on " The Settlement of London due to Underdrainage of the London Clay." [2] The most recent Paper that he presented to the Institution was on " The Bearing Capacity of Screw Piles and Screwcrete Cylinders." [3] Dr Wilson was the Author of many other Papers on soil mechanics and foundation engineering.

He was also a member of the Institution of Structural Engineers, and of the American and French Societies of Civil Engineers.

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[1] J. Instn Civ. Engrs, vol. 17, p. 87 (Nov. 1941).
[2] J. Instn Civ. Engrs, vol. 19, p. 100 (Dec. 1942).
[3] J. Instn Civ. Engrs, vol. 34, p. 4 (Mar. 1950).

CORRIGENDUM

Proceedings, Part I, March 1953—

p. 155, line 2. *After* "December, 1952," *add* "and the 20th January, 1953,"

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